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16. Abstract

Based on a recent change in Texas Department of Transportation (TxDOT) policy, frontage roads are not to be included along controlled-access highways unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. Interchange design options that do not include frontage roads are to be considered for all new freeway construction. Ramps in non-frontage-road settings can be more challenging to design than those in frontage-road settings for several reasons. Adequate ramp length, appropriate horizontal and vertical curvature, and flaring to increase storage area at the crossroad intersection should all be used to design safe and efficient ramps for non-frontage-road settings. However, current design procedures available in standard TxDOT reference documents focus on ramp design for frontage-road settings. The objective of this research project was to develop recommended design procedures for interchange ramps on facilities without frontage roads.

This report describes the findings from the second year of a two-year project. Crash data from Texas interchanges were used to calibrate several ramp safety prediction models. These models form the basis for a procedure for evaluating the safety of alternative ramp configurations. Ramp design guidelines are developed. Design controls and elements routinely considered during the ramp design process are identified. The report titled *Recommended Ramp Design Procedures for Facilities without Frontage Roads* presents this guidance in a manner suitable for application by designers.

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DEVELOPMENT OF RAMP DESIGN PROCEDURES FOR FACILITIES WITHOUT FRONTAGE ROADS

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DISCLAIMER

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NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1. INTRODUCTION

OVERVIEW

Based on a recent change in TxDOT policy (Minute Order 108544), frontage roads are not to be included along controlled-access highways (i.e., freeways) unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. The intent of this policy change is to extend the service life of the freeway and the mobility of the corridor by promoting development away from the freeway. Interchange design options that do not include frontage roads are to be considered for all new freeway construction.

Ramps in non-frontage-road settings can be more challenging to design than those in frontage-road settings for two reasons. First, they must provide drivers a safe transition between the high-speed freeway and the stop condition at the crossroad intersection. Unlike the main lanes, a ramp's design speed changes along its length such that ramp length and design speed change are interrelated. Ramp curves must be carefully sized such that speed changes along the ramp occur in safe and comfortable increments for both cars and trucks.

Second, ramp design for non-frontage-road settings is challenging because the "effective" ramp length (i.e., that portion of the ramp measured from the gore area to the back of queue) can vary based on traffic demands. Thus, during peak demand hours, the speed change may need to occur over a relatively short length of ramp. In contrast, the speed change can occur over the full length of the ramp during low-volume conditions. Sound ramp design should accommodate such variation in effective ramp length by conservatively designing for the high-volume condition. Similar issues exist for entrance ramp design when ramp metering or high-occupancy-vehicle (HOV) bypass lanes are present.

In summary, adequate ramp length, appropriate horizontal and vertical curvature, and flaring to increase storage area at the crossroad intersection might be used to design operationally superior ramps for non-frontage-road settings. However, current design procedures available in standard TxDOT reference documents focus on ramp design for frontage-road settings. Therefore, a need exists to develop interchange ramp design procedures (or modify existing guidelines) that address the aforementioned concerns and that can be used to construct safe and efficient interchange ramps for facilities without frontage roads.

RESEARCH OBJECTIVE

The objective of this research project was to develop recommended design procedures for interchange ramps on facilities without frontage roads. The procedures developed describe the operational and safety benefits of alternative ramp configurations. They also offer guidelines that help engineers select the most appropriate configuration for design-year traffic. The procedures are based on the findings from a review of the ramp design guidelines used by other state departments of transportation (DOT), the analysis of data obtained from field and simulation studies, and the analysis of crash data for interchanges in Texas.

RESEARCH SCOPE

The guidelines developed for this research are applicable to the geometric design of interchanges in urban, metro, and rural environments on Texas freeways without frontage roads. The guidelines address controls and considerations for designing the ramp proper and ramp terminal of both exit and entrance ramps. The guidelines reflect consideration of the performance and physical aspects of both passenger cars and trucks. Finally, they also reflect a sensitivity to safety and operations. The research does not address the question of "Where or when should frontage roads be used?" This question is appropriately addressed by the TxDOT administration (and the Texas Transportation Commission) and is a matter of agency policy.

The guidelines address two types of service interchange commonly used in non-frontage-road settings. They are the diamond and partial cloverleaf (or "parclo") interchanges. Typical variations of both types are shown in Figure 1-1.

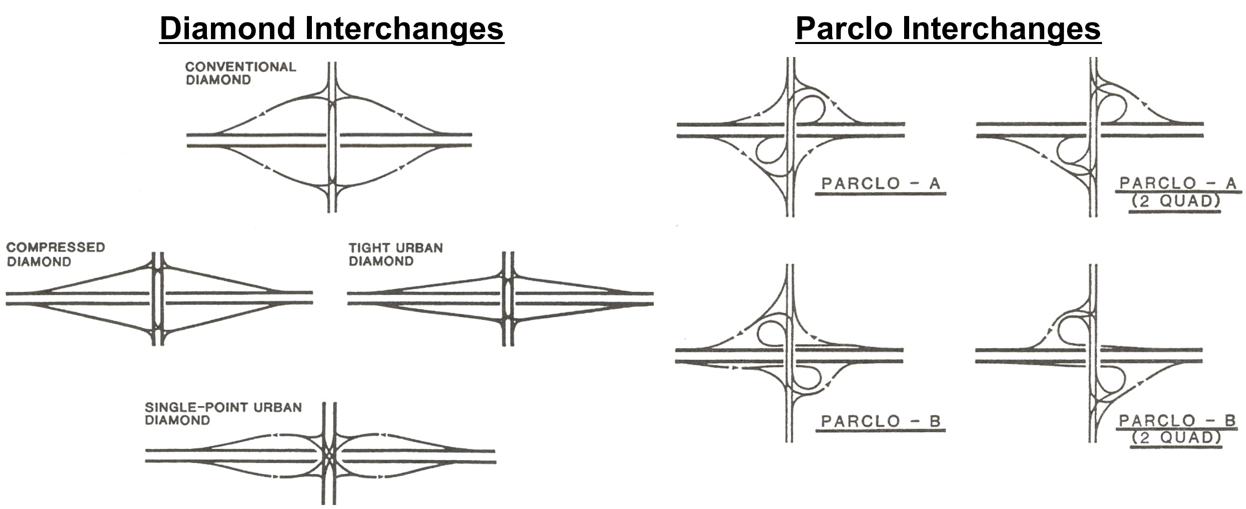


Figure 1-1. Interchange Types Commonly Used in Non-Frontage-Road Settings.

The parclo interchanges are much more common in states not having frontage roads. The parclo A and parclo B types have two loop ramps that each serve one turn movement and ramps in all four quadrants. In contrast, the 2-quad parclo variations serve two turn movements on each loop ramp and, thereby, minimize right-of-way requirements in two quadrants. The generous allocation of ramps at the parclo A and parclo B results in a high capacity for all interchanging movements.

On the other hand, the 2-quad parclo variations tend to have fewer ramps but at the cost of lower traffic carrying capacity.

RESEARCH APPROACH

The project's research approach is based on a two-year program of field study, evaluation, and development. This approach has produced a guideline document to assist in the design of interchange ramps on facilities without frontage roads. During the first year of the project, the state-of-the-practice was defined and the operational effects of several ramp design configurations were evaluated. An analytic procedure was developed for evaluating and comparing the operations of alternative ramp configurations. Report 0-4538-1 *Review and Evaluation of Interchange Design Considerations for Facilities without Frontage Roads* documents the findings from the state-of-the-practice review and the ramp evaluation procedure.

The research conducted in the second year of the project is documented herein. This research included an evaluation of the safety effects of alternative ramp configurations. The findings from this evaluation were used to develop a procedure for evaluating and comparing the safety of alternative ramp configurations. They were also used as a basis for the ramp design guidelines.

The main product of this research is Report 0-4538-3, *Recommended Ramp Design Procedures for Facilities without Frontage Roads*. This report provides technical guidance for engineers who desire to design safe and efficient interchange ramps on facilities without frontage roads. It also provides guidelines for selecting the most appropriate ramp configuration based on operational and safety considerations. The analytic procedures in the Report 0-4538-3 are implemented in an Excel ® spreadsheet. The spreadsheet is available from the report authors.

CHAPTER 2. DEVELOPMENT OF A RAMP SAFETY EVALUATION PROCEDURE

OVERVIEW

This chapter documents the development of a procedure for estimating the safety of alternative ramp configurations. The procedure is based on two safety prediction models calibrated using the crash data reported at several Texas interchanges. The calibration process is consistent with that used by other researchers to calibrate safety prediction models for intersections (*1, 2*).

In general, safety prediction models relate crash frequency at a facility to its traffic flow, traffic control, and design-related characteristics. They have several uses in safety analysis, including: (1) identifying facilities that may benefit from one or more safety improvements, and (2) providing numerical tools for evaluating alternative design configurations. The safety prediction models described in this chapter are envisioned to be particularly useful during the concept planning and preliminary design stages of the design process, where they can be used to identify cost-effective interchange design configurations.

This chapter is organized into four main sections. The first section summarizes previous research on interchange-related safety prediction models and issues related to their calibration for local conditions. The second section describes the data collection plan and summarizes the study site crash characteristics. The third section describes the model calibration process and the recommended calibration factors. The fourth section describes a procedure for using the calibrated models to evaluate the safety benefit of alternative ramp configurations.

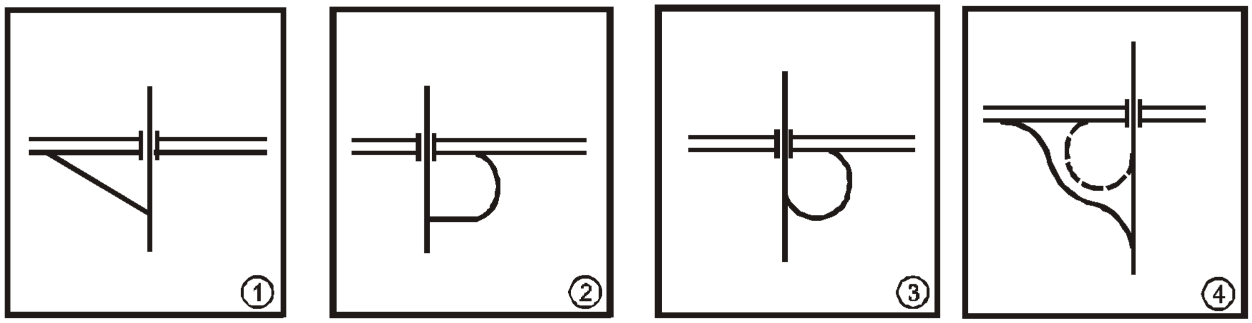
REVIEW OF RAMP SAFETY LITERATURE

Safety Prediction Models

Four ramp configurations are commonly used at interchanges without frontage roads; they are:

- diagonal,
- non-free-flow loop,
- free-flow loop, and
- outer connection.

Exit ramp variations of each configuration are illustrated in Figure 2-1. Entrance ramp versions have a similar alignment. The safety record of these four ramp configurations is the subject of discussion in this section.



a. Diagonal. **b. Non-Free-Flow Loop.** **c. Free-Flow Loop.** **d. Outer Connection.**

Figure 2-1. Basic Ramp Configurations at Non-Frontage-Road Interchanges. (*3*)

Interchange safety has been the focus of several previous research projects. The findings from these projects were examined by Bonneson et al. (*4*). In general, these studies focused on the safety of individual ramps as opposed to the overall interchange. This focus was due to the unique influence of ramp design on crash potential. The findings from their examination indicated that:

- Exit ramps have 35 to 65 percent more crashes than entrance ramps.
- For similar traffic demands, free-flow loop ramps tend to have the fewest crashes; diagonal ramps have slightly more, outer connection ramps have still more, and the non-free-flow loop ramps have the most crashes.
- Ramp length was not found to have a significant effect on crash frequency.
- Ramps in urban areas have about 40 percent more crashes than those in rural areas.

The only safety prediction models specific to ramp type and configuration found in the literature are those developed by Bauer and Harwood (*3*). Their models were developed with data obtained from the State of Washington for the years 1993 to 1995. One of these models predicts the frequency of crashes of all severities (i.e., property-damage-only, injury, and fatal). The form of this model is:

$$N_t = 0.247\ C_t\, f_t \left(\frac{V_r}{1000}\right)^{0.76} \qquad \textbf{(1)}$$

where,

N_t = predicted annual number of crashes (of all severities), crashes/yr;
f_t = crash adjustment factor for area type, ramp type, and ramp configuration (see Table 2-1);
C_t = calibration factor for local conditions; and
V_r = average daily traffic on the ramp, veh/d.

The subscript t associated with each model variable denotes that the variable represents crashes of all severities. The standard deviation of the predicted annual number of crashes can be estimated as:

$$s_{Nt} = \frac{N_t}{\sqrt{k_t}} \qquad \textbf{(2)}$$

where,

k_t = dispersion parameter for crashes of all severities (= 0.95); and

s_{Nt} = standard deviation of the predicted annual number of crashes (of all severities), crashes/yr.

The adjustment factor f_t in Equation 1 is used to adapt the model to alternative combinations of area type, ramp type, and ramp configuration. The regression analysis reported by Bauer and Harwood (*3*) indicated that the factors listed in columns 3 and 7 in Table 2-1 are appropriate for this purpose. The adjustment factors in columns 4 and 8 are discussed in subsequent paragraphs.

Table 2-1. Crash Adjustment Factors for Equations 1 and 3.

Area Type: Rural				Area Type: Urban			
Ramp Type	Ramp Configuration	Adjustment Factor		Ramp Type	Ramp Configuration	Adjustment Factor	
		f_t	f_{f+i}			f_t	f_{f+i}
Exit	Diagonal	1.00	1.00	Exit	Diagonal	1.42	1.40
	Non-free-flow loop	1.75	1.97		Non-free-flow loop	2.48	2.77
	Free-flow loop	0.63	0.59		Free-flow loop	0.89	0.83
	Outer connection	1.31	1.30		Outer connection	1.86	1.82
Entrance	Diagonal	0.61	0.58	Entrance	Diagonal	0.86	0.81
	Non-free-flow loop	1.06	1.14		Non-free-flow loop	1.51	1.60
	Free-flow loop	0.38	0.34		Free-flow loop	0.54	0.48
	Outer connection	0.79	0.75		Outer connection	1.13	1.05

The adjustment factors f_t in Table 2-1 vary from 0.38 to 2.48, depending on the combination of attributes associated with a specific ramp. The range is relatively wide and suggests that ramp design decisions can have a significant impact on safety. The relationship between crash frequency and ramp traffic demand for exit ramps at rural interchanges, as predicted by Equation 1, is shown in Figure 2-2.

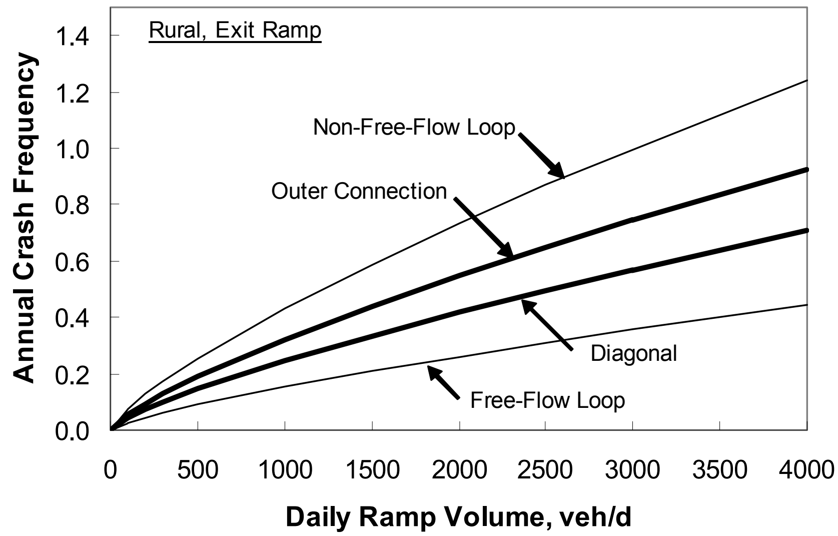


Figure 2-2. Effect of Ramp Volume on Total Crash Frequency at Rural Exit Ramps.

A second model developed by Bauer and Harwood (*3*) predicts the number of fatal and injury crashes. The form of this model is:

$$N_{f+i} = 0.0957\ C_{f+i}\, f_{f+i} \left(\frac{V_r}{1000} \right)^{0.85} \tag{3}$$

where,

N_{f+i} = predicted annual number of fatal and injury crashes, crashes/yr;
f_{f+i} = crash adjustment factor for area type, ramp type, and ramp configuration (see Table 2-1);
C_{f+i} = calibration factor for local conditions; and
V_r = average daily traffic on the ramp, veh/d.

The subscript f+i associated with each model variable denotes that the variable represents injury and fatal crashes (i.e., property-damage-only crashes are *not* included). The standard deviation of the predicted number of fatal and injury crashes can be estimated as:

$$s_{Nf+i} = \frac{N_{f+i}}{\sqrt{k_{f+i}}} \tag{4}$$

where,

k_{f+i} = dispersion parameter for fatal and injury crashes (= 0.70); and
s_{Nf+i} = standard deviation of the predicted annual number of fatal and injury crashes, crashes/yr.

The adjustment factors f_{f+i} in Equation 3 are provided in columns 4 and 8 of Table 2-1. They vary from 0.34 to 2.77, depending on the combination of attributes associated with a specific ramp. The relationship between severe crash frequency and ramp traffic demand for exit ramps at rural interchanges, as predicted by Equation 3, is shown in Figure 2-3.

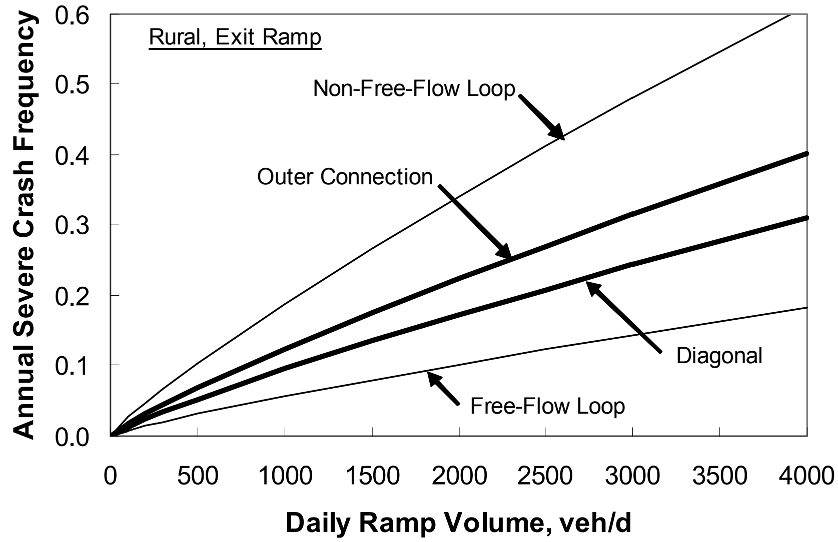


Figure 2-3. Effect of Ramp Volume on Severe Crash Frequency at Rural Exit Ramps.

The adjustment factors C_t and C_{f+i} in Equations 1 and 3, respectively, can be used to adapt the model to locations outside of the state of Washington. The process used to determine their value for a specific locale is described in the next section.

Issues Related to Model Calibration for Local Conditions

Ideally, robust safety prediction models (such as those developed by Bauer and Harwood *(3)* and Harwood et al. *(5)*) would be developed for every state and local jurisdiction responsible for the design and operation of highway facilities. Such models would accurately account for differences in crash causation among jurisdictions (e.g., due to differences in terrain, design practices, climate, trip purpose, or crash reporting threshold). However, the cost of developing these models would be prohibitively expensive, primarily because of the effort required to assemble the database needed for each model's development. In recognition of this practical limitation, the robust models that have been developed for a few jurisdictions are typically converted for use by other jurisdictions through a calibration process.

Models and data associated with the jurisdictions for which they were initially developed are referred to herein as "base models" and "base crash data." Data associated with the jurisdictions to which the models are extended through calibration are referred to as "local crash data."

The calibration process described by Harwood et al. (*5*) is generally recognized as being appropriate for safety prediction models. It is based on the following assumptions: (1) the variables in the base model are also correlated with local crash data, (2) no additional variables are needed in the base model to explain the variability in local crash data, (3) the calibration coefficients in the base model accurately reflect the relationship between model variables and local crash frequency (although there may be some bias relative to local conditions); and (4) a single, multiplicative calibration factor can be used to eliminate the aforementioned bias by uniformly increasing (or decreasing) the crash frequency estimate obtained from the base model.

MODEL DEVELOPMENT

Three options for developing a safety prediction model for application to ramps in Texas were identified and evaluated. These options include:

1. Develop a ramp safety prediction model using Texas data.
2. Calibrate a valid existing ramp safety prediction model using Texas data.
3. Develop an interchange safety prediction model using Texas data.

All three of these options are based on the use of crash data from interchange ramps in Texas. They are listed in order of desirability from the standpoint of prediction accuracy; with Option 1 being the most desirable. The feasibility and benefit of each option are discussed in the following paragraphs.

Option 1 - Develop Ramp Models Using Texas Data

The model developed from this option would predict the number of crashes associated with a specific interchange ramp configuration. It would require data for about 100 ramps of each configuration (i.e., 400 in total). The model developed from this effort would accurately reflect the effects of traffic demand and geometry on ramp crashes in Texas. It would also reflect Texas weather, terrain, reporting practices, and motorist behavior. As such, it would be more accurate than the models developed from the other options.

Several challenges would have to be overcome before this option could be pursued with success. These challenges relate to the need for an average annual daily traffic volume (AADT) estimate for each ramp and the accurate identification of ramp-related crashes. Unfortunately, ramp AADTs are not routinely collected in Texas. Also, the attributes used to describe crash location in the Texas Department of Public Safety (DPS) crash database are not sufficiently well-defined to ensure that all ramp-related crashes can be reliably identified. These two limitations can be overcome by: (1) conducting a short-term traffic count at each ramp, (2) obtaining printed copies of the crash reports for the most recent three years at the corresponding interchanges, and (3) manually screening the printed reports for ramp-related crashes. The resources required to assemble the volume and crash data for this option would be significant.

Option 2 - Calibrate Valid Existing Ramp Models Using Texas Data

Option 2 provided a more reasonable balance between the resources available and those required to gather the needed AADT and crash data. As with Option 1, the model developed would predict the number of crashes associated with a specific interchange ramp. This option would require data for about 10 to 25 ramps of each configuration (40 to 100 in total).

The calibration process described by Harwood et al. (*5*) would be used with this option to calibrate Equations 1 and 3. It requires ramp AADT and crash data for a relatively small sample of interchange ramps. As noted previously, the volume data are not readily available in Texas and would require the conduct short-term counts at each of the ramps represented in the database. Also, the printed crash reports for these same interchanges would need to be acquired and manually screened. These data would be needed for a smaller sample of ramps, relative to Option 1.

A drawback of this option is that the regression coefficients in Equations 1 and 3, as well as those in Table 2-1 (identified as "adjustment factors"), are not changed as a result of this calibration technique. Only the calibration factors C_t and C_{f+i} are changed to reflect Texas conditions. Given that the regression coefficients were calibrated using data from the State of Washington, the calibrated versions of Equations 1 and 3 may be less accurate that those obtained from Option 1. This possibility was noted by Persaud et al. (*1*) in an examination of the accuracy of intersection safety prediction models developed in one state and calibrated for use in another state.

Option 3 - Develop Interchange Models Using Texas Data

Option 3 was developed to overcome the data limitations outlined in the discussion of Option 1. The objective of this option is to identify diamond and parclo interchanges with typical diagonal, loop, and outer connector ramp configurations. The model developed with this option would predict the total number of crashes associated with an interchange having a specified combination of ramps. This number would include both ramp-related and non-ramp-related crashes, hence, it could not be used to directly predict ramp crash frequency. However, the difference in total crashes for two interchange types, where the only difference was ramp configuration, would be an estimate of the crashes due to the different ramp configurations.

This option was developed in recognition of two facts: (1) highway AADTs are readily available in the Texas DPS crash database, and (2) this database can be used to identify crashes within a specified distance from the point of intersection of the two highways. Thus, to overcome the lack of ramp AADTs in the database, the AADTs for each of the intersecting highways would be used instead of the ramp AADTs. To overcome the inability to accurately identify ramp-related crashes, the database to be assembled for model development would consist of all crashes on the major road that occur within ±1000 ft of the interchange plus all those that occur on the minor road within ±500 ft of the interchange.

Detailed maps of the Texas highway system and a database of aerial photographs were examined to determine the number of interchanges suitable for use with this option. Four criteria were used: (1) the interchanges could not have frontage roads, (2) the interchanges had to have either a diamond, parclo A, parclo B, parclo A (2-quad), or parclo B (2-quad) form, (3) the ramp configuration had to be "symmetric" such that diagonally opposed quadrants had the same ramp configuration, and (4) the two intersecting highways had to be on the state highway system. After a careful review of the entire state highway system, no interchanges of the parclo variety could be found that satisfied the four criteria.

Recommended Option

As the discussion in the preceding section indicated, three options for developing safety prediction models were evaluated. Options 1 and 2 are challenged by the lack of ramp traffic volume data and the lack of precision in the Texas DPS crash database. Option 3 was rendered infeasible by the lack of suitable parclo interchanges in Texas. Based on these findings, it was decided that Option 2 would be pursued. This option required completion of the following steps:

1. Identify a sample of interchange ramp configurations.
2. Collect AADTs at each of the ramps.
3. Acquire printed crash reports for each interchange.
4. Manually screen the crash reports to identify the ramp-related crashes.
5. Calibrate Equations 1 and 3 using the ramp-related crash data.

The details of the data collection activities associated with Steps 1, 2, 3, and 4 are described in the next section. The activities associated with Step 5 are described thereafter.

DATA COLLECTION PLAN

This section describes the approach undertaken for the collection of data suitable for the calibration of Equations 1 and 3. Initially, the various types of data included in the database are identified. Then, the interchange ramps selected for study are described. Next, the methods used to collect the various types of data are discussed. Finally, the characteristics of the data collected are outlined in the last section. Henceforth, a study "site" is defined to be one interchange ramp.

Database Composition

The composition of the database was dictated by the variables included in the safety prediction models to be calibrated (i.e., Equations 1 and 3). Collectively, these variables describe the crash characteristics of each ramp as well as its geometric and traffic characteristics. The variables needed to calibrate Equations 1 and 3 are listed in Table 2-2. Data representing each variable were collected for each of 44 ramps at 10 interchanges in Texas. The location of these ramps is described in a subsequent section.

Table 2-2. Safety Database Composition.

Category	Variable	Variable Values	Source
Crash	Severity	Property-damage-only, injury, fatal	DPS
	Crash type	Ran-off-road, rear-end, sideswipe	
	Relationship	Associated with ramp (excl. speed-change lanes and ramp terminal)	
Ramp Geometry	Type	Exit, entrance	Aerial photos
	Configuration	Diagonal, non-free-flow loop, free-flow loop, outer connection	
Environment	Area type	Rural, urban	
Traffic	Ramp AADT	Vehicles	Field survey

Crash data were extracted from printed peace officer crash reports of selected crashes that occurred in the vicinity of the interchange. These reports were obtained from the Texas DPS for the most recent three years for which data were available (i.e., 1998, 1999, and 2000). Aerial photographs of each interchange were also obtained and used to identify ramp configuration and area type.

Study Site Characteristics

Several criteria were used to identify the ramp study sites represented in the database. These criteria include:

- Each ramp must be located at an interchange without frontage roads.
- The set of ramps must collectively represent the four configurations shown in Figure 2-1.
- The set of ramps must collectively represent urban and rural areas.
- The set of ramps must collectively represent entrance and exit ramp types.

The sites included in the database are listed in Table 2-3. All total, 44 ramps at 10 interchanges are represented in the database. These interchanges are located in TxDOT's Austin District. An aerial photograph of each interchange is provided in the appendix.

The ramps selected for inclusion in the database and their locations were strongly influenced by the level of resources available to the project. It was established that a minimum of 10 ramps (desirably 25 ramps) of each configuration would be needed for statistical significance. However, the lack of available ramp AADTs and the need for a manual review of the officer crash reports dictated the commitment of considerable time to the acquisition of data for each site. As a consequence of this reassignment of resources, the number of ramps included in the database did not exceed the minimum needed for model calibration.

Table 2-3. Candidate Study Sites.

County	Address	Terminal Control[1]	Area Type	No. of Sites for Each Ramp Configuration[2, 3]			
				Diagonal	NFF Loop	FF Loop	Outer
Bastrop	US 290 & SH 21	TWSC	Rural	4	0	0	0
Caldwell	US 183 & SH 21	TWSC	Rural	4	0	0	0
Lee	SH 21 & US 77	TWSC	Rural	4	0	0	0
Travis	Loop 360 & RM 2222	Signal	Urban	4	0	0	0
	US 183 & FM 969	Signal	Urban	4	0	0	0
	US 183 & SH 71	Free	Rural	0	0	3	4
	SH 1 & Windsor	Signal	Urban	1	2	0	0
	SH 1 & 35th Street	Sig./TWSC	Urban	1	1	2	2
	Loop 360 & RM 2244	Signal	Urban	4	0	0	0
Williamson	US 79 & Business 79	TWSC	Rural	0	0	2	2
			Total Candidate Sites:	26	3	7	8

Notes:
1 - TWSC: two-way stop control; Signal: traffic control signal; Free: merge from a parallel or tapered ramp terminal.
2 - NFF Loop: non-free-flow loop; FF Loop: free-flow loop.
3 - A "site" is one interchange ramp.

Data Collection Process

The data collection process focused on the acquisition of two types of data: ramp crashes and ramp AADTs. The methods used to collect and reduce these data are described in the next two sections.

Crash Data Collection Process

The crash data collection process included the following steps:

1. Identify all crashes occurring in the vicinity of the interchanges listed in Table 2-3 for the years 1998, 1999, and 2000 using the Texas DPS crash database.
2. For all crashes identified in Step 1, request copies of the printed crash reports from the DPS.
3. Manually review each report and determine if the crash is ramp-related.

For Step 1, a crash was defined to be in the "vicinity" of the interchange when it occurred on the major road within at least ±1000 ft of the interchange center or when it occurred on the minor road within at least ±500 ft of the interchange center. The interchange "center" was defined by the major-road and minor-road mile points that correspond to the intersection of the two highway alignments. Using these definitions, a total of 586 crashes were found in the vicinity of the 10 interchanges. A request was submitted to the DPS for all 586 crash reports.

For Step 3, ramp-related crashes were identified in the set of 586 reports. A ramp-related crash was defined as any crash where the vehicle causing the crash was traveling on the ramp just prior to the crash. Crashes that occurred on a free-right-turn lane between the ramp proper and minor road were considered to be ramp related. However, crashes that occurred in a speed-change lane (i.e., gore area) or within the intersection at the ramp terminal were not considered to be related to the ramp.

Ramp AADT Data Collection Process

The AADT for each ramp was estimated using a one-day (i.e., 24-hour) count taken in January or February 2004. The AADT was estimated from the one-day count by adjusting it for day-of-week and month-of-year variations. The day-of-week and month-of-year adjustment factors were obtained from TxDOT's Transportation Planning and Programming (TPP) Division and are based on their 2003 Automated Traffic Record (ATR) database. Two criteria were used to select the specific ATR recorders considered for this development. First, the recorder used should be as close as possible to the subject interchange. Second, the functional class of the roadway on which the recorder is located should be the same as that of the major road associated with the subject interchange.

The AADT of the major road was obtained for the years 1998, 1999, and 2000 from AADT maps developed by TPP. The set of three AADTs for each interchange were used to compute an annual traffic growth adjustment factor. This factor was then used to convert the 2004 ramp counts into an equivalent 1999 AADT.

Database Characteristics

Table 2-4 summarizes the data collected for the calibration process. As indicated in the first row of data, only 43 sites are represented in the database. One site (i.e., the northbound exit ramp at Loop 360 and RM 2222) was found to have unusual crash patterns. In particular, this ramp was found to have experienced 24 crashes during the three-year study period. This frequency is about nine times larger than the average crash frequency of the other 43 sites. Closer examination indicated that the ramp had the following attributes: (1) relatively sharp crest curve, (2) excessive queues, and (3) a public street intersection just downstream of the crest curve. It is believed that these attributes combine to explain many of the crashes that occurred. This ramp was removed from the database because driveways and intersections are rarely provided on ramps in Texas. It was the only ramp with an intersection located along its length.

The data in Table 2-4 indicate that 38 crashes occurred on the set of 43 ramp study sites during the three-year study period. Of these crashes, 21 were associated with a fatality or injury to one or more persons. In other words, 55 percent of the crashes were fatal or injury-related and 45 percent were property-damage-only (PDO) crashes. The latter percentage is smaller than the nationwide average of 58 percent PDO crashes, as derived from Table 28 of *Traffic Safety Facts*

2000 (*6*). This trend is likely due to the use of a higher crash reporting threshold in the various Texas jurisdictions, relative to those jurisdictions reflected in *Traffic Safety Facts 2000*.

Table 2-4. Summary of Crash and Traffic Volume Database.

Category	Statistic [1]	Ramp Type		Ramp Configuration [2]				Area Type		Overall
		Exit	Ent-rance	Dia-gonal	NFF Loop	FF Loop	Outer	Rural	Urban	
Total Sites		21	22	25	3	7	8	23	20	43
All Crashes	Min. crashes/3 yr	0	0	0	1	0	0	0	0	0
	Max. crashes/3 yr	5	3	5	3	4	2	4	5	5
	Average crashes/3 yr	1.14	0.64	0.64	2.33	1.29	0.75	0.74	1.05	0.88
	Total crashes/3 yr	24	14	16	7	9	6	17	21	38
	Crash Rate [3], cmv	**0.26**	**0.16**	**0.15**	**0.26**	**0.45**	**0.20**	**0.45**	**0.15**	**0.21**
Fatal & Injury Crashes	Min. crashes/3 yr	0	0	0	1	0	0	0	0	0
	Max. crashes/3 yr	3	2	3	3	1	1	1	3	3
	Average crashes/3 yr	0.67	0.32	0.40	2.00	0.29	0.38	0.30	0.70	0.49
	Total crashes/3 yr	14	7	10	6	2	3	7	14	21
	Crash Rate [3], cmv	**0.15**	**0.08**	**0.10**	**0.22**	**0.10**	**0.10**	**0.18**	**0.10**	**0.12**
1999 Ramp Volume	Minimum, veh/d	250	100	250	5450	100	300	100	2200	100
	Maximum, veh/d	10500	10000	9600	10500	7450	10000	10000	10500	10500
	Average, veh/d	4000	3700	3800	8200	2600	3400	1500	6500	3850

Notes:
1 - Statistics reflect the crash history for years 1998, 1999, and 2000.
2 - NFF Loop: non-free-flow loop; FF Loop: free-flow loop.
3 - cmv: crashes per million vehicles.

The statistics listed in Table 2-4 indicate that a total of 38 crashes were reported at the 43 interchange ramps studied. This total equates to an average of 0.88 crashes for each ramp during a three-year period (or 0.29 crashes/ramp/yr). One ramp experienced a maximum of five crashes during a three-year period. However, twenty-two ramps did not experience any crashes in a three-year period.

The crash rates shown in Table 2-4 are useful to identify general trends in the data and to facilitate comparison with previous research. With regard to ramp type, exit ramps are found to have about 62 percent more crashes (= 0.26/0.16 − 1) than entrance ramps, given the same ramp traffic volume. This trend is consistent with that found by Bauer and Harwood (*3*) and Twomey et al. (*7*). The former researchers reported that exit ramps have 65 percent more crashes whereas the latter group reported 35 percent more crashes.

With regard to ramp configuration, the crash rates shown in Table 2-4 indicate that free-flow loop ramps are associated with about twice as many crashes as other ramps. This finding is contrary to that found in the safety relationships reported by Bauer and Harwood (*3*) and by Twomey et al. (*7*). Both of these groups of researchers found that free-flow loop ramps were associated with the *fewest* crashes. This trend is not apparent in the crash rates for the "fatal and injury" crashes. For this crash category, the *non*-free-flow loop is associated with the highest rate of severe crashes; a trend that is consistent with Bauer and Harwood (*3*).

Setting aside the free-flow loop ramp, the remaining three ramp categories can be listed in order of decreasing crash rate as: non-free-flow loop, outer connection, and diagonal ramp. This order is consistent with that found in the safety relationships reported by Bauer and Harwood (*3*) and by Twomey et al. (*7*). Therefore, while the crash rate for free-flow loops in Texas is higher than expected, the other trends related to ramp configuration are consistent with reported trends. It is possible that the high crash rate for free-flow loops in Texas is related to driver familiarity. These loops are not used as often in Texas as in the states included in the data examined by Bauer and Harwood or by Twomey et al.

With regard to area type, the crash rates in Table 2-4 indicate that rural ramps have significantly more crashes than urban ramps, given the same traffic volume. This trend is logical given the higher speeds found at rural interchanges and their lack of safety lighting. However, this finding is contrary to that reported by Bauer and Harwood (*3*). They found that rural ramps were associated with fewer crashes. Further examination of the data in Table 2-4 indicates that the crash rate for rural ramps is very consistent with that reported by Twomey et al. (*7*). Thus, it is possible that the urban ramps included in this study have an exceptionally low crash risk, relative to the urban ramps in Washington (as studied by Bauer and Harwood), or that many of the PDO crashes on urban ramps are not being reported to the DPS.

Closer examination of the distribution of crash severity indicates that the PDO percentage varies for the rural and urban crashes. For rural crashes, it is 59 percent which is consistent with the nationwide average. For urban ramps, it is only 33 percent. This finding suggests that only about 36 percent of the PDO crashes occurring on the urban ramps are being reported. It also partially explains the difference between rural and urban crash rates discussed in the previous paragraph. It suggests that the urban crashes listed in the "all crashes" category in Table 2-4 should be inflated by 1.60 ($= \{1-0.33\}/\{1-0.58\}$) to reflect a more typical distribution of fatal, injury, and PDO crashes.

MODEL CALIBRATION

Calibration Process

The methods proposed by Persaud et al. (*1*) and others (*2,5*) were used for the calibration process. However, instead of one calibration factor for all combinations of area type, ramp type, and ramp configuration, separate calibration factors were considered for each attribute combination. Separate analyses were conducted for the "all crash" and the "fatal and injury crash" models.

Several statistical tests were used to determined which attribute combination offered the best fit between the calibrated model and the crash data. Based on these tests, it was determined that separate calibration factors for urban and rural ramps would provide the most accurate safety prediction model.

The calibration factor for rural ramps $C_{t,\,rural}$ was estimated with the following equation:

$$C_{t,rural} = \frac{\sum_i O_{t,rural,i}}{n \sum_i N_{t,rural,i}} \left(1 + \frac{Var[N_{t,rural}]}{\left(\sum_i N_{t,rural,i} \right)^2} \right)^{-1} \qquad \textbf{(5)}$$

where,

$C_{t,rural}$ = calibration factor for all crashes on rural ramps;
$O_{t,rural,i}$ = observed crashes (of all severities) on rural ramp i during n years, crashes;
$N_{t,rural,i}$ = predicted annual number of crashes (of all severities) on rural ramp i; crashes/yr;
n = number of years crashes were observed (n = 3 for the ramp database); and
$Var[x]$ = variance of random variable x.

The variance of the predicted annual number of crashes $Var[N_{t,rural}]$ was estimated using the following equation:

$$Var[N_{t,rural}] = \sum_i \frac{(N_{t,rural,i})^2}{k_t} \qquad \textbf{(6)}$$

where,

k_t = dispersion parameter for crashes of all severities (= 0.95).

Equations 5 and 6 were also used to compute $C_{t,urban}$ for the urban ramps. In this instance, the observed and predicted crashes corresponded to the urban ramps. The dispersion factor k_t reported by Bauer and Harwood (*3*) of 0.95 was used in Equation 6.

Equations 5 and 6 were also used to compute calibration factors for the severe crashes (i.e., fatal and injury). As with total crashes, separate calibration factors were estimated for rural and urban ramps $C_{f+i,rural}$ and $C_{f+i,urban}$. A dispersion factor k_{f+i} of 0.70 was used.

The second term in Equation 5 (i.e., the term raised to the power -1) was used for this calibration because of the relatively small number of crashes in the ramp database. Hauer (*8*) recommends using this term to remove the bias associated with the first term when the database includes fewer than 500 crashes. For the ramp database, this bias adjustment term reduced the quantity obtained from the first term by about 10 percent.

The variance of $C_{t,rural}$ is given by:

$$Var[C_{t,rural}] = C_{t,rural}^2 \left(\frac{Var[O_{t,rural}]}{\left(\sum_i O_{t,rural,i}\right)^2} + \frac{Var[N_{t,rural}]}{\left(\sum_i N_{t,rural,i}\right)^2} \right) \left(1 + \frac{Var[N_{t,rural}]}{\left(\sum_i N_{t,rural,i}\right)^2} \right)^{-2} \quad \textbf{(7)}$$

The variance of the observed number of crashes $Var[O_{t,rural}]$, as needed for Equation 7, was estimated using the following equation:

$$Var[O_{t,rural}] = \sum_i O_{t,rural,i} \quad \textbf{(8)}$$

This equation is based on the assumption that the observed crash counts follow a Poisson distribution. Equation 8 was used to compute the variance of $C_{t,urban}$, $C_{f+i,rural}$, and $C_{f+i,urban}$, with proper substitution of the applicable observed and predicted crash data.

The calibration factors obtained from Equations 5 through 8 are shown in Table 2-5. This table also shows the standard error for each factor. The standard error is an indicator of the level of uncertainty in the corresponding calibration factor.

Table 2-5. Calibration Factors.

Statistic	Area Type: Rural		Area Type: Urban	
	All Crashes	Fatal & Injury	All Crashes	Fatal & Injury
ΣO	17	7	21	14
$n \ \Sigma N$	18.8	7.6	78.0	36.3
k	0.95	0.70	0.95	0.70
Var[N]	3.9	1.0	47.4	15.3
Calibration Factor, *C*	**0.83**	**0.80**	**0.25**	**0.35**
Standard Error of C	0.30	0.38	0.08	0.13

Calibrated Safety Prediction Model

The calibration factors in Table 2-5 were combined with the adjustment factors in Table 2-1 to produce a single calibration constant for use in simplified versions of Equations 1 and 3. The resulting constants for the "all crashes" category for urban ramps were also inflated by a factor of 1.60 to account for the apparent under-reporting of urban ramp crashes in Texas. The resulting calibration coefficients are listed in Table 2-6.

The simplified safety prediction model for crashes of all severities is:

$$N_t = 0.247\ a_t \left(\frac{V_r}{1000} \right)^{0.76} \tag{9}$$

where,

N_t = predicted annual number of crashes (of all severities), crashes/yr;
a_t = model calibration coefficient for area type, ramp type, and ramp configuration; and
V_r = average daily traffic on the ramp, veh/d.

Table 2-6. Calibrated Model Coefficients.

Area Type: Rural				Area Type: Urban			
Ramp Type	Ramp Configuration	Model Coefficient		Ramp Type	Ramp Configuration	Model Coefficient	
		a_t	a_{f+i}			a_t	a_{f+i}
Exit	Diagonal	0.83	0.80	Exit	Diagonal	0.57	0.49
	Non-free-flow loop	1.45	1.58		Non-free-flow loop	0.99	0.97
	Free-flow loop	0.52	0.47		Free-flow loop	0.35	0.29
	Outer connection	1.09	1.04		Outer connection	0.74	0.64
Entrance	Diagonal	0.50	0.46	Entrance	Diagonal	0.34	0.28
	Non-free-flow loop	0.88	0.91		Non-free-flow loop	0.60	0.56
	Free-flow loop	0.31	0.27		Free-flow loop	0.22	0.17
	Outer connection	0.66	0.60		Outer connection	0.45	0.37

The subscript *t* associated with each model variable in Equation 9 denotes that the variable represents crashes of all severities (including property-damage-only, injury, and fatal crashes). The standard deviation of the predicted annual number of crashes is:

$$s_{Nt} = 1.03\ N_t \tag{10}$$

The simplified safety prediction model for fatal and injury crashes is:

$$N_{f+i} = 0.0957\ a_{f+i} \left(\frac{V_r}{1000} \right)^{0.85} \tag{11}$$

where,

N_{f+i} = predicted annual number of fatal and injury crashes, crashes/yr.

The subscript f+i associated with each model variable in Equation 11 denotes that the variable represents injury and fatal crashes (i.e., property-damage-only crashes are *not* included). The standard deviation of the predicted annual number of fatal and injury crashes is:

$$s_{Nf+i} = 1.20\, N_{f+i} \qquad \textbf{(12)}$$

PROCEDURE FOR COMPARING ALTERNATIVE INTERCHANGE TYPES

This section describes a procedure for comparing alternative interchange types and ramp configurations in terms of their expected crash frequency. The procedure is based on the models described previously in this report. The procedure can be used to predict the expected annual crash frequency for each ramp which can then be aggregated to obtain an estimate for the entire interchange. Crashes that occur in the vicinity of the speed-change lanes (i.e., gore area) and those that occur within the ramp terminal conflict area are not considered in this procedure.

General Procedure

The procedure described herein is suitable for the concept planning and preliminary design stages of an interchange project. It can be used to obtain a quick estimate of the expected crash frequency associated with a particular interchange type or ramp configuration for specified ramp AADTs. The procedure consists of four steps. These steps are described in the following sections.

Step 1. Identify Area Type and Movement AADTs

Area type and interchange turn movement AADTs should be identified in this step. Area type is categorized as "urban" or "rural." It is based on the land use surrounding the interchange. Urban land uses are typically characterized as "developed" with access to this development provided on the cross street in the immediate vicinity of a ramp terminal. The development can be residential, commercial, industrial, or business. In urban areas, interchange spacing along the major road is typically less than 2.0 miles. Interchanges in areas that are not consistent with these urban characteristics should be considered as "rural" interchanges.

The design-year AADT for each interchange turn movement is needed for the analysis. The turn movements of interest are the left-turn and right-turn onto and off of each ramp. This distinction is not essential for some ramp configurations (e.g., diagonal); however, it is needed to evaluate other configurations (e.g., free-flow loop) that serve only one turn movement at the interchange. Whenever possible, design-year turn movement AADTs should be obtained from the planning division. However, if such estimates are unavailable, then the technique described in the section titled "Interchange Turn Movement Estimation Technique" can be used to estimate them.

Step 2. Identify Candidate Ramp Configurations for Each Interchange Quadrant

For this step, the ramp configuration for each quadrant should be identified and matched to the appropriate safety prediction model. The configurations for which a model exists include:

- diagonal ramp,
- non-free-flow loop ramp,
- free-flow loop ramp, and
- outer connection ramp.

The location of a ramp's major-road speed-change lane (i.e., gore area) defines the quadrant in which the ramp is located. This definition is intuitive for diagonal and outer connection ramps because their entire length is located in the same quadrant. However, it is not as intuitive in the case of loop ramps. For these ramps, good design practice is to locate the major-road speed-change lane upstream of the interchange for exit ramps and downstream of the interchange for entrance ramps. When this practice is followed, the "loop" portion of a loop ramp is not located in the same quadrant as its speed-change lane.

In most cases, the specification of a ramp configuration for a specific quadrant directly follows from the interchange type being considered (i.e., diamond, parclo A, parclo A [2-quad], parclo B, parclo B [2-quad]). For example, the diagonal exit ramp is typically used for the diamond, parclo A, and parclo A (2-quad) interchanges. Similarly, the diagonal entrance ramp is typically used for the diamond, parclo B, and parclo B (2-quad). A non-free-flow loop is used for the parclo B (2-quad) exit ramp and for the parclo A (2-quad) entrance ramp. These ramp configurations serve both a left-turn and a right-turn volume (as identified in Step 1). These two volumes would be summed to estimate the ramp AADT.

For the parclo A entrance ramp and parclo B exit ramp, the outer connection ramp is typically used in combination with a free-flow loop ramp. The outer connection serves the right-turn movement while the free-flow loop serves a left-turn movement (in the context of the change in travel direction made by traveling through the interchange). In both cases, the ramp AADT is the same as the AADT of the one turn movement that it serves. This AADT would be used in the appropriate safety prediction model to estimate ramp crash frequency.

Occasionally, the diamond, parclo A, and parclo A (2-quad) interchanges may have a more generous ramp design that resembles the combination of a diagonal ramp and an outer connection. This "combined" ramp design is shown in Figure 2-4. The ramp shown illustrates the geometry of an exit ramp; however, the combined ramp design is also applicable to entrance ramps. A technique for estimating the safety of this ramp configuration is described in the section titled "Combined Ramp Configuration."

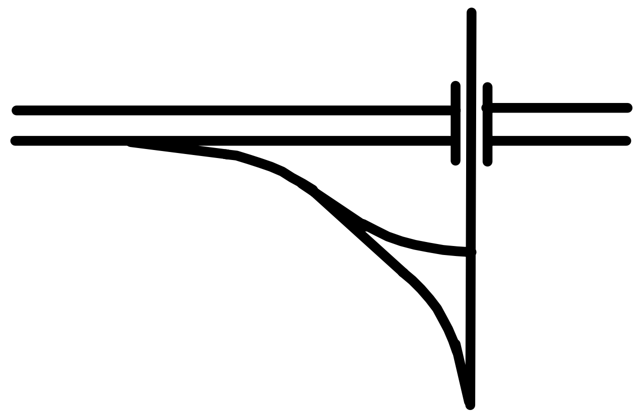


Figure 2-4. Combined Diagonal and Outer Connection Ramp Design.

Step 3. Estimate Annual Crash Frequency for Each Ramp Alternative

For this step, the annual crash frequency is estimated for each ramp alternative considered. The frequency of "all crashes" (i.e., PDO, injury, or fatal) as well as the frequency of severe crashes (i.e., injury or fatal) can be separately computed using Equations 9 and 11, respectively. These equations should be used in combination with the calibrated model coefficients provided in Table 2-6. Techniques for estimating ramp crash frequency are described in a section titled "Crash Frequency Estimation."

The frequency of severe crashes should be given particular consideration because ramp alternatives that have the fewer severe crashes are likely to be more cost-effective to construct, all other considerations being equal. The frequency of "all crashes" is useful in selecting the safer ramp configuration when two or more ramp configurations have similar severe crash frequencies. In this situation, the ramp configuration with the lower frequency of "all crashes" represents the safer configuration.

Step 4. Estimate Interchange Crash Frequency

For this step, the crash frequencies for selected ramp configurations can be aggregated into an overall interchange crash frequency. This step does not need to be conducted unless the overall crash frequency is desired. The frequency of all crashes, severe crashes, or both can be aggregated.

The aggregation of ramp crash frequencies requires the specification of compatible ramp configurations for each ramp quadrant. For example, the diamond interchange would require specification of diagonal ramps in each of the four quadrants. A parclo A would require specification of diagonal exit ramps in two quadrants and a free-flow loop entrance ramp plus outer connection entrance ramp in the other two quadrants.

The interpretation of the overall interchange crash frequency is consistent with that used for the individual ramps. Interchange alternatives that have fewer severe crashes are likely to be more cost-effective to construct, all other considerations being equal. The frequency of "all crashes" is useful in selecting the safer interchange type when two or more types have similar severe crash frequencies. In this situation, the interchange type with the lower frequency of "all crashes" represents the safer configuration.

Interchange Turn Movement Estimation Technique

If design-year turn movements are not available, they can be estimated using the AADT of the major road and the technique described in this section. The percentages listed in Table 2-7 can be used for this purpose. These percentages are based on an examination of the AADTs of the 44 ramps used in the safety database.

Table 2-7. Interchange Turn Movement Volumes as a Percentage of Major-Road AADT.

Ramp Configuration	Turn Movement Percentages by Area Type, %	
	Urban	Rural
Diagonal, Non-free-flow loop [1]	8.0	18.0
Free-flow loop, Outer connection	4.0	9.0

Note:

1 - Percentages listed for the diagonal and non-free-flow loop ramps relate to the total ramp volume (i.e., they include both left-turn and right-turn volumes). For these ramp configurations, the left-turn volume is estimated as 50 percent of the total ramp volume. The remaining volume represents that of the right-turn movement.

The percentages listed in Table 2-7 can be multiplied by the major-road AADT to estimate the corresponding ramp AADT. The diagonal and non-free-flow loop ramps serve both left-turn and right-turn volumes, hence, the estimated ramp AADT must be further multiplied by the left-turn (or right-turn) percentage to determine the left-turn AADT (or right-turn AADT) for the ramp. The left-turn volume is estimated as 50 percent of the total ramp volume. In contrast, the free-flow loop and outer connection ramp configurations serve either a left-turn or a right-turn volume (but not both). Hence, the ramp AADT estimated from Table 2-7 for free-flow loop and outer connection ramps also represents a turn movement AADT.

To illustrate the technique, consider an interchange located in an urban area with a major-road AADT of 50,000 veh/d. A parclo A is being considered for this location. A diagonal exit ramp is used with this interchange type. Table 2-7 indicates that the exit ramps for this interchange would likely serve about 8.0 percent of the major-road AADT, or 4000 veh/d. Of this amount, 50 percent will turn left at the ramp terminal and the other 50 percent will turn right. Thus, the exit ramp left-turn AADT is 2000 veh/d and the exit ramp right-turn AADT is 2000 veh/d.

Continuing the illustration, the parclo A has a free-flow loop ramp and an outer connection ramp for movements entering the major road. The free-flow loop ramp effectively serves as a left-turn movement at the interchange (i.e., drivers make a left turn in their direction of travel). Its AADT can be estimated as 4.0 percent of the major-road AADT. The outer connection ramp serves as a right-turn movement. Its AADT can also be estimated as 4.0 percent of the major-road AADT. Thus, the cross street left-turn AADT is 2000 veh/d and the cross street right-turn AADT is 2000 veh/d.

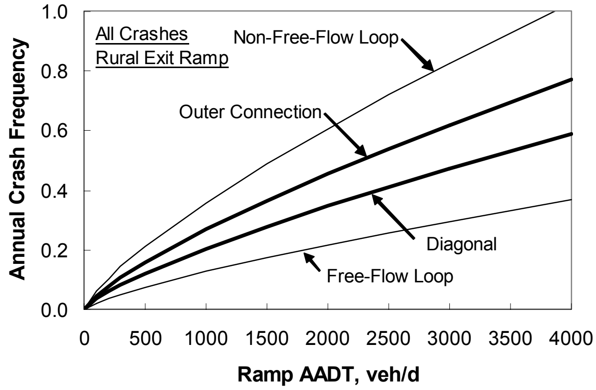
Combined Ramp Configuration

The "combined" ramp configuration is occasionally used at some diamond and parclo interchanges. It represents a combination of the outer connection and diagonal ramps. Its geometry was previously shown in Figure 2-4. The geometry and operation of the combined ramp includes the best operational features of the diagonal and outer connector ramps. The left-turn movement is served at the intersection, and the right-turn movement is served by a turning roadway. A disadvantage of this design is that it requires considerable right-of-way and significant distance between the interchange and the nearest downstream intersection on the crossroad. As such, it is best-suited to rural locations.

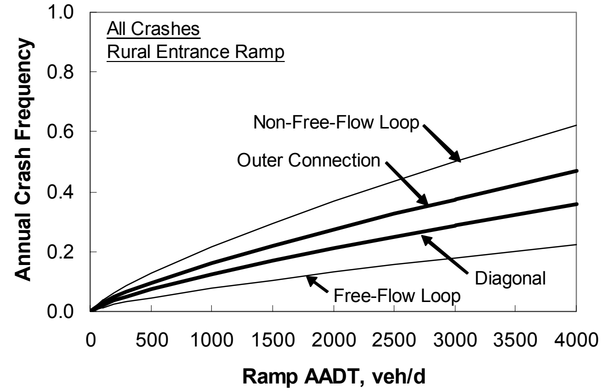
A safety prediction model is not available for the combined ramp configuration. However, the models developed for the diagonal ramp and the outer connection ramp can be used to estimate crash frequency for the combined ramp. In this application, both models would be used and their estimates of crash frequency combined. For the diagonal ramp model, the AADT used would be that of the left-turn movement for the ramp. For the outer connection model, the AADT used would be that of the right-turn movement. The two crash frequencies (one from each model) would then be summed to obtain an estimate of the combined ramp crash frequency.

Crash Frequency Estimation

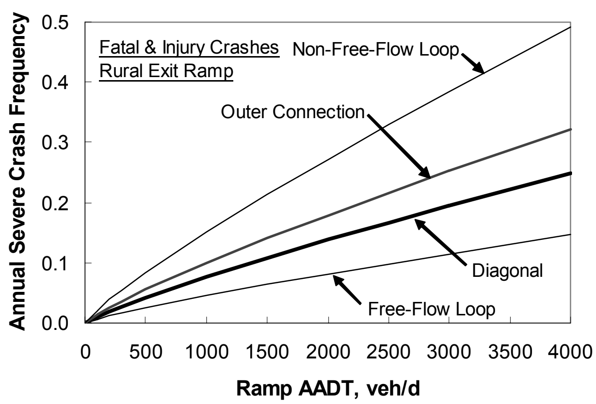
Equations 9 and 11 are calibrated for use in computing the expected crash frequency for interchange ramps located in Texas. They are applicable only to interchanges in non-frontage-road settings. They can be used to evaluate the safety of alternative ramp configurations for an interchange proposed for construction or an interchange undergoing major reconstruction. The equations should be recalibrated every three years to ensure that they continue to reflect current driver behavior and design practices in Texas. Figures 2-5 and 2-6 can also be used to graphically estimate crash frequency. They were developed using Equations 9 and 10 for a reasonable range of AADTs.



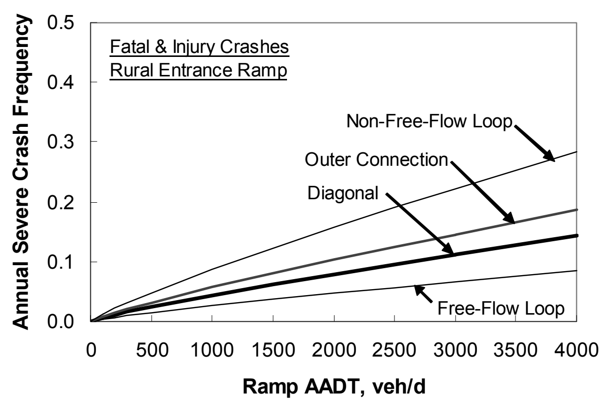
a. All Crashes on Exit Ramps.



b. All Crashes on Entrance Ramps.

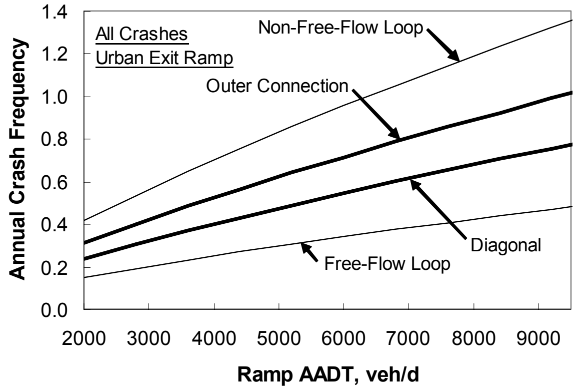


c. Severe Crashes on Exit Ramps.

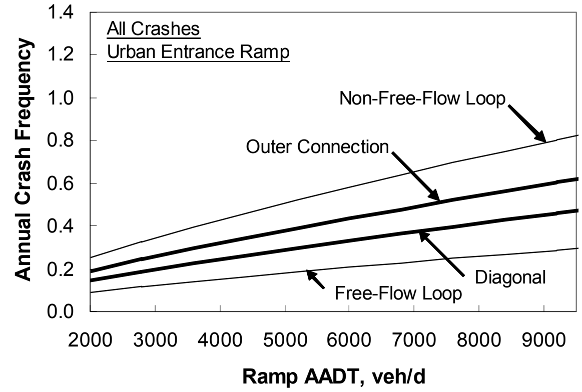


d. Severe Crashes on Entrance Ramps.

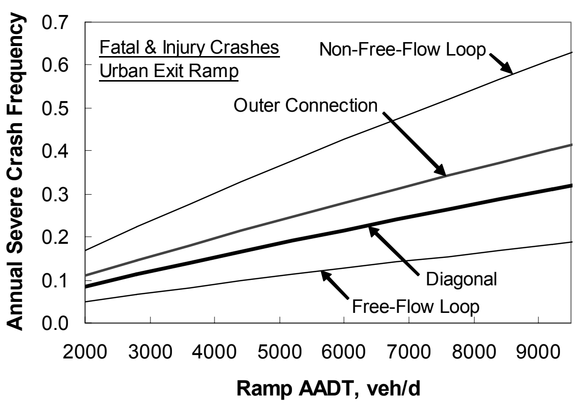
Figure 2-5. Rural Ramp Crash Frequency.



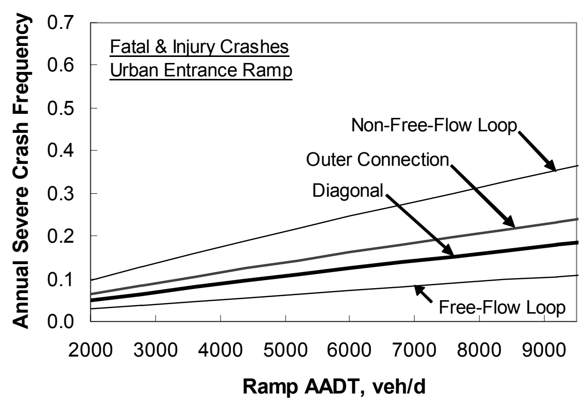
a. All Crashes on Exit Ramps.



b. All Crashes on Entrance Ramps.



c. Severe Crashes on Exit Ramps.



d. Severe Crashes on Entrance Ramps.

Figure 2-6. Urban Ramp Crash Frequency.

CHAPTER 3. DEVELOPMENT OF RAMP DESIGN GUIDELINES

OVERVIEW

This chapter describes the development of ramp design procedures for freeway facilities without frontage roads. Design controls and elements routinely considered during the ramp design process are identified. The procedures are organized to be consistent with Chapter 3, Section 6, of the *Roadway Design Manual* (*9*). The discussion associated with each design control and element is intended to provide a rational basis for its use as well as guidance on its application. This guidance is presented in a more concise manner in the document titled *Recommended Ramp Design Procedures for Facilities without Frontage Roads*.

The focus of this chapter is on the design controls and elements applicable to ramps in non-frontage-road settings. Design controls and elements that are common to ramps in both frontage and non-frontage-road settings are addressed in the *Roadway Design Manual* (*9*) and in *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*10*) and are not repeated herein.

The first section to follow this overview describes interchange types used with freeway facilities without frontage roads. It provides some guidance on the selection of ramp configuration based on consideration of traffic demands, topographic features, and right-of-way constraints. It also provides context for the discussion of the ramp design controls and elements that follow.

The second section describes the use of design speed to define key elements of ramp design. The approach is tailored to the ramp's configuration and function such that design speed changes are gradual and consistent with driver expectancy.

The third section describes the design controls and elements that dictate the horizontal geometry of both entrance and exit ramps. Controls considered include: vertical curvature, maximum superelevation rate, minimum radius, queue storage length, speed-change length, and superelevation transition length.

The fourth section addresses two elements of the ramp cross section. Initially, the controls and considerations that guide in the selection of the number of lanes on the ramp proper are discussed. Thereafter, controls related to the ramp traveled-way width are described.

The last section focuses on ramp terminal design. Design controls addressed include: intersection skew angle, approach cross section, and storage length. The selection of an appropriate intersection traffic control mode (i.e., stop or signal control) is also discussed. Access control limits along the crossroad are also described.

SERVICE INTERCHANGES

The *Green Book* indicates that there are two basic categories of interchange: system interchanges and service interchanges. System interchanges serve all turning movements without traffic control and, hence, are used for the intersection of two freeway facilities. In contrast, service interchanges have some type of stop or signal control for one or more left-turn movements. These interchanges are most appropriate at locations where the intersecting facility is classified as a local, collector, or arterial roadway. Service interchanges are much more frequent in number than system interchanges and are the subject of this report.

Alternative Interchange Types

The two most common types of service interchange for non-frontage-road settings are the diamond and parclo. Typical variations of both types are shown in Figure 1-1. The absence of frontage roads allows for the consideration of a wide range of interchange types. This range allows for a more cost-effective ramp configuration to accommodate site-specific traffic demands, topographic features, and right-of-way constraints. The absence of frontage roads eliminates the U-turn traffic movement inherent to frontage road interchanges. A high-volume U-turn movement often causes operational problems at the interchange if it is not provided an exclusive U-turn lane.

The diamond and parclo interchanges can be further categorized by their ramp separation distance, ramp geometry, ramp control mode, and crossroad cross section. With respect to these categories, the attributes of typical diamond interchanges are listed in Table 3-1; those of typical parclos are listed in Table 3-2.

As indicated in Table 3-1, one advantage of the diamond interchange is that the turn movements from the major road and from the crossroad are "true" to the intended change in direction of travel. In other words, a driver makes a left turn at the interchange when desiring to make a left turn in travel direction. This characteristic is desirable because it is consistent with driver expectancy. Unfamiliar drivers can be confused if a loop ramp configuration requires them to make a right turn at the interchange when they desire to make a left turn in their direction of travel.

In urbanized areas, the tight urban diamond interchange (TUDI) and the single-point urban interchange (SPUI) can provide efficient traffic operation along the crossroad. The TUDI's ramp terminals are easily coordinated using a single signal controller. This ease of coordination is due primarily to the interchange's relatively short ramp separation distance. The efficiency of the SPUI stems from its use of a single signalized junction and non-overlapping left-turn paths. In contrast, the compressed diamond is not as operationally efficient as the TUDI or the SPUI. This characteristic is due to the compressed diamond's wider ramp separation, which is not as conducive to crossroad signal coordination. As a result, the compressed diamond interchange is better suited to rural or suburban settings where traffic demands are low to moderate.

Table 3-1. Characteristics of Typical Diamond Interchanges.

Category		Diamond Interchange Type [1, 2]			
		Conventional	**Compressed**	**TUDI**	**SPUI**
Ramp Separation (centerline to centerline)		800 to 1200 ft	400 to 800 ft	200 to 400 ft	150 to 250 ft (stopline to stopline)
Typical Location		Rural	Suburban	Urban	Urban
Ramp Terminal Control		● 2 stop signs ● 2 actuated signals	● 1 actuated signal ● 2 semi-act. signals	● 1 actuated signal ● 1 or 2 pretimed signals	● 1 actuated signal
Crossroad Left-Turn Bay Geometry	Location	Internal to terminals.	Internal to terminals.	Internal and possibly external, if needed.[3]	**External to intersection.**
	Length	200 to 300-ft bay.	150 to 300-ft bay.	Parallel bays, if needed.[3]	As needed for storage.
Signal Coordination		**Often not essential but can be achieved.**	Often needed but difficult to obtain.	**Needed and easily achieved.**	**Single signal.**
Volume Limits		Moderate.	Moderate.	**Moderate to high.**	**Moderate to high.**
Bridge Width		**Through lanes only.**	Through lanes plus width of median and often part of both left-turn bays.	Through lanes plus both left-turn bays, if needed.[3]	Through lanes plus width of median and wider left-turn lane.
Operational Experience		**Acceptable.**	Acceptable–sometimes the need for progression is a problem.	**Acceptable.**	**Acceptable.**
Signal Phases/Terminal		3, if signalized	3	3	3
Left from Crossroad		**Via left turn.**	**Via left turn.**	**Via left turn.**	**Via left turn.**
Left from Major Road		**Via left turn.**	**Via left turn.**	**Via left turn.**	**Via left turn.**
Right from Crossroad		**Via right turn.**	**Via right turn.**	**Via right turn.**	**Via right turn.**
Right from Major Road		**Via right turn.**	**Via right turn.**	**Via right turn.**	**Via right turn.**
Queues on Exit-Ramp?		Yes	Yes	Yes	Yes

Notes:
1 - Table content based on information provided in References *11*, *12*, and *13*.
2 - Characteristics in **bold** font are generally recognized as advantageous in terms of operations, safety, or cost.
3 - If left-turn and U-turn demands are low to moderate and four-phase operation is provided, then bays are generally not needed. If left-turn or U-turn demands are high or four-phase operation is not provided, then left-turn bays are needed between the ramp terminals. If left-turn bays are provided, then they should extend backward through the upstream ramp terminal.

The parclos shown in Figure 1-1 are most applicable to situations where a specific left-turn movement pair has sufficiently high volume to have a significant negative impact on ramp terminal operation. Both variations of the parclo A provide uncontrolled service for crossroad drivers intending to turn left (in travel direction) at the interchange. Both variations of the parclo B typically provide uncontrolled service for major-road drivers intending to turn left at the interchange.

Table 3-2. Characteristics of Typical Partial Cloverleaf Interchanges.

Category		Parclo Interchange Type [1, 2]			
		Parclo A	**Parclo B**	**Parclo A (2-quad)**	**Parclo B (2-quad)**
Ramp Separation [3] (centerline to centerline)		700 to 1000 ft	1000 to 1400 ft	700 to 1000 ft	1000 to 1400 ft
Typical Location		Suburban	Suburban	Rural	Rural
Ramp Terminal Control		● 2 semi-actuated	● 2 semi-actuated	● 2 stop signs ● 2 actuated signals	● 2 stop signs ● 2 actuated signals
Crossroad Left-Turn Bay Geometry	Location	**Not applicable.**	Internal to terminals.	**External to terminals.**	Internal to terminals.
Crossroad Left-Turn Bay Geometry	Length	Not applicable.	$\leq$ 40 percent of ramp separation distance.	Based on volume.	$\leq$ 40 percent of ramp separation distance.
Signal Coordination		**Often needed and easily achieved.**	**Not needed as downstream through is unstopped.**	Rarely needed in rural settings.	Rarely needed in rural settings.
Volume Limits		**Moderate to high.**	**Moderate to high.**	Moderate.	Moderate.
Bridge Width		**Through lanes only.**	Through lanes plus width of median.	**Through lanes only.**	Through lanes plus width of median.
Operational Experience		**Acceptable.**	**Acceptable.**	Potential for wrong-way movements.	Potential for wrong-way movements.
Signal Phases/Terminal		2	2	3	3
Left from Crossroad		Via right turn.	**Via left turn.**	Via right turn.	**Via left turn.**
Left from Major Road		**Via left turn.**	Via right turn.	**Via left turn.**	Via right turn.
Right from Crossroad		**Via right turn.**	**Via right turn.**	Via left turn.	**Via right turn.**
Right from Major Road		**Via right turn.**	**Via right turn.**	**Via right turn.**	Via left turn.
Queues on Exit-Ramp?		Yes (on diagonal)	**No**	Yes (on diagonal)	Yes (on loop)

Notes:
1 - Table content based on information provided in References *11* and *13*.
2 - Characteristics in **bold** font are generally recognized as advantageous in terms of operations, safety, or cost.
3 - Ramp separation distances listed for the parclo A and parclo A (2-quad) are based on 170-ft loop radii (25-mph design speed). Distances listed for the parclo B and parclo B (2-quad) are based on 250-ft loop radii (30-mph design speed).

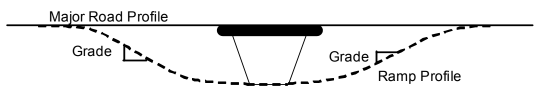
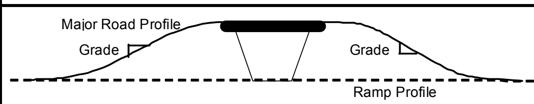
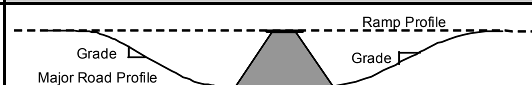
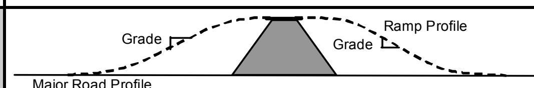
The parclo A and parclo B are more efficient than their "2-quad" counterparts. This feature stems from their elimination of one left-turn movement from the ramp terminal signalization. One advantage of the parclo A is that it satisfies the expectancy of major-road drivers by providing turn movements that are "true" to the driver's intended direction of travel. A second advantage is that it does not require left-turn bays on the crossroad. This advantage can result in a narrower bridge. A third advantage is that its terminal design is not conducive to wrong-way movements. Finally, the speed change from the crossroad to the loop ramp is likely to be relatively small and easy to accommodate with horizontal curves of minimum radius.

The parclo B also has several advantages. One advantage is that its signalized ramp terminals do not require coordination because signal timing for the outbound travel direction can provide a continuous green indication. A second advantage is that it does not require queues to form on the exit ramp because the left-turn and right-turn movements are unsignalized at their intersection with the crossroad. Finally, its ramp terminal design is not conducive to wrong-way movement.

Overpass vs. Underpass

A fundamental consideration in interchange design is whether the major road should be carried over (i.e., an overpass design) or under the crossroad. When topography does not govern, the relative advantages and disadvantages listed in Table 3-3 should be considered when selecting an overpass or underpass design *(10)*. They also provide some insight as to the merit of locating the crossroad below, at, or above the existing ground level.

Table 3-3. Advantages of the Overpass and Underpass Configurations.

Crossroad Location Relative to Existing Ground	Major Road Location Relative to Crossroad	
	Overpass	**Underpass**
Below	 ● Offers best sight distance along major road.	Not applicable.
At	 ● Offers best possibility for stage construction. ● Eliminates drainage problems.	 ● Reduced traffic noise to adjacent property. ● Provides best view of ramp geometry.
Above	Not applicable.	 ● Ramp grades decelerate exit-ramp vehicles and accelerate entrance-ramp vehicles. ● Eliminates drainage problems. ● Typically requires least earthwork.
	<u>Other Overpass Advantages:</u> ● Through traffic is given aesthetic preference. ● Accommodates oversize loads on major road.	<u>Other Underpass Advantages:</u> ● Interchange and ramps easily seen by drivers on the major road. ● Bridge size (for crossroad) is smaller.

The information in Table 3-3 identifies advantages of the overpass and underpass designs. However, it appears that the underpass design offers greater benefit when ramp safety and operations are key considerations. The underpass design with the crossroad elevated "above" ground level is often the most advantageous because it provides major-road drivers with: (1) ample preview distance as they approach the interchange, and (2) ramp grades that are helpful in slowing exit-ramp drivers and accelerating entrance-ramp drivers. The one exception to this generalization is the SPUI. For this interchange, the underpass design with the crossroad “at” ground level is preferred because it provides the driver the best view of the ramp geometry in the terminal area. This view is important at a SPUI because of its unusual ramp terminal design.

Selection Considerations

Interchange selection for a specific location generally requires consideration of a wide range of factors. When focusing on operation and right-of-way requirements, the interchange forms most amenable to specific combinations of facility class and area type are listed in Table 3-4.

Table 3-4. Interchange Types Amenable to Various Facility Classes and Area Types.

Intersecting Street or Highway Classification	Area Type		
	Rural	Suburban	Urban
Local	Conventional Diamond Parclo A (2-quad) Parclo B (2-quad)	Compressed Diamond	Tight Urban Diamond
Collector or Arterial	Parclo A Parclo B	Parclo A Parclo B	Single-Point Urban Diamond Tight Urban Diamond

Only the more common interchange types are listed in Table 3-4. Other interchange types or geometric variations of the types listed may be appropriate in specific situations. Directional ramps may be added to one or more interchange quadrants to serve a specific high-volume turn movement.

Interchange selection should reflect consideration of safety, operation, uniformity of exit patterns (relative to adjacent interchanges), cost, availability of right-of-way, potential for stage construction, and compatibility with the environment *(10)*. The selection of interchange type for rural areas is based primarily on traffic demand, especially turn movement demands. In urban areas, selection is based on traffic demands, interchange spacing, and right-of-way impacts. Interchanges with loop ramps can be very efficient at locations with heavy left-turn volumes; however, their right-of-way requirements can preclude them from consideration in built-up urban environments.

Procedures for evaluating the safety and operation of interchange types have been developed for this research. The procedures are sufficiently general that they can be used for the selection of ramp configuration and interchange type for the concept planning and preliminary design stages of the design process. The safety evaluation procedure is described in Chapter 2. The operations evaluation procedure is described by Bonneson et al. *(4)*.

DESIGN SPEED

This section describes the use of design speed as a control in ramp design. The approach is tailored to the ramp's configuration such that design speed changes are gradual and consistent with driver expectancy and operational capabilities. It is based on the specification of reasonable speed changes along the various tangents and curves that compose the ramp's horizontal alignment. The total amount of speed change needed is dictated by the design speed of the major road and that of the intersecting crossroad or ramp terminal. This approach is consistent with the ramp design speed guidance provided on page 829 of the *Green Book* (*10*).

The first subsection identifies the tangent and curve segments that compose the typical interchange ramp alignment. The second subsection identifies a reasonable design speed for each segment. The segments and speeds identified in these two subsections are applicable to most ramps; however, they may not be applicable in certain specific situations. In these situations, the methodology is sufficiently general that it can be used to identify segments and corresponding design speeds for any ramp alignment. Where unusual ramp geometry is encountered, design speeds that differ from those suggested by this approach can be specified without compromising the validity of the controls described in later sections of this chapter.

Ramp Segments

This section describes the individual road segments that compose the horizontal alignment for the interchange ramp. These segments consist of both tangents and curves. The ramp configurations addressed are those applicable to service interchanges without frontage roads and include: diagonal, loop, and outer connection ramps. Segments are identified for both the exit and entrance ramp of each configuration. The figures in this section are not drawn to scale and the segment lengths and locations shown may not reflect the proportions found in actual design.

The segments that compose the diagonal ramp are shown in Figure 3-1. Both the exit and entrance ramps consist of three tangents and two curves. For the exit ramp, Tangent 1 provides for initial vehicle deceleration to the design speed of the controlling curve (i.e., Curve 1). Tangent 2 provides a length for transitioning the superelevation between Curves 1 and 2. Tangent 3 provides a length of roadway for deceleration and storage associated with the ramp terminal. The segments for the entrance ramp perform a similar function but in reverse order to those of the exit ramp.

Although the details of major-road speed-change lane (i.e., gore area) design are not the subject of this report, the divergence and convergence angles associated with this design do influence the design of the ramp alignment and are discussed for this purpose. The divergence angle of the exit ramp between Tangent 1 and the major road should range from two to five degrees (*10*) with a typical value being three degrees (*14*). The convergence angle of the entrance ramp between Tangent 3 and the major road is based on the taper rate used. Typical values range from 50:1 to 70:1. This range equates to a convergence angle of about one degree.

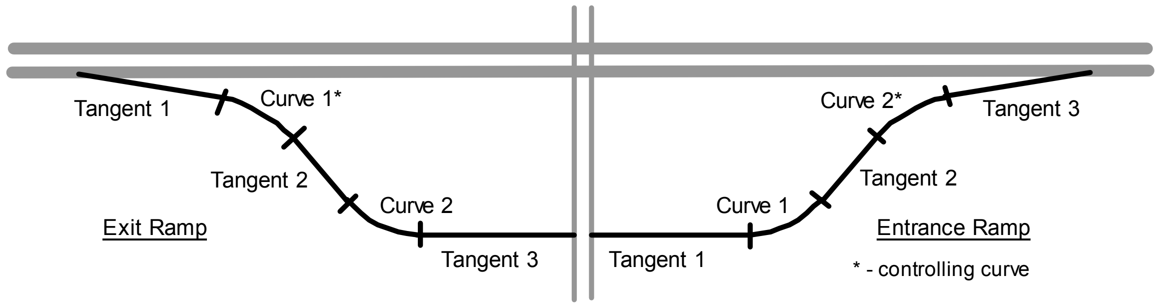


Figure 3-1. Diagonal Ramp Segments.

Segments for a parclo B loop exit ramp are shown in Figure 3-2. The alignment shown includes three curves. Curves 1 and 2 are intended to promote driver awareness of the impending loop (i.e., Curve 3) and encourage drivers to gradually reduce speed prior to their arrival to Curve 3. The deflection angles for Curves 1 and 2 are each about two to three times larger than the divergence angle. The location of Tangent 3 varies depending on whether a parclo B or parclo B (2-quad) ramp is selected. If desired, a spiral curve equal in length to 2.0-s travel time at the design speed can be located between Curves 2 and 3.

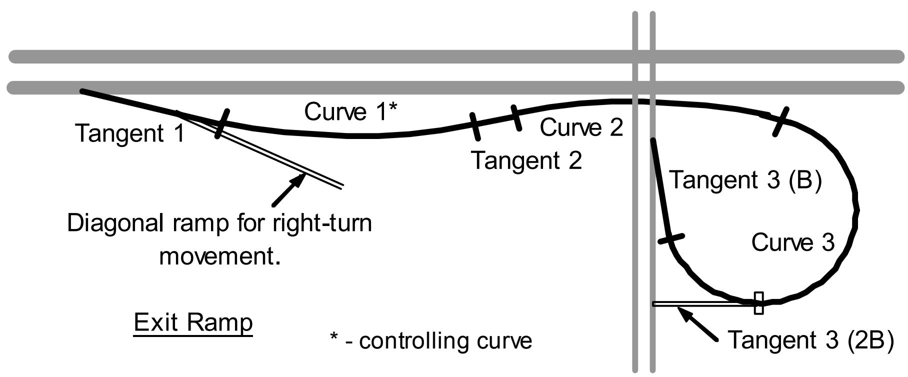


Figure 3-2. Loop Exit Ramp Segment.

Segments for a parclo A loop entrance ramp are shown in Figure 3-3. The alignment includes a short segment of tangent to transition from the crossroad (or ramp terminal) design speed to that of the loop ramp. The location of this segment varies depending on whether a parclo A or parclo A (2-quad) is selected for design. The length of Tangent 2 is based on the distance needed to accelerate from the loop design speed to that of the major road. If desired, a spiral curve equal in length to 2.0-s travel time at the design speed can be located between Curve 1 and Tangent 2.

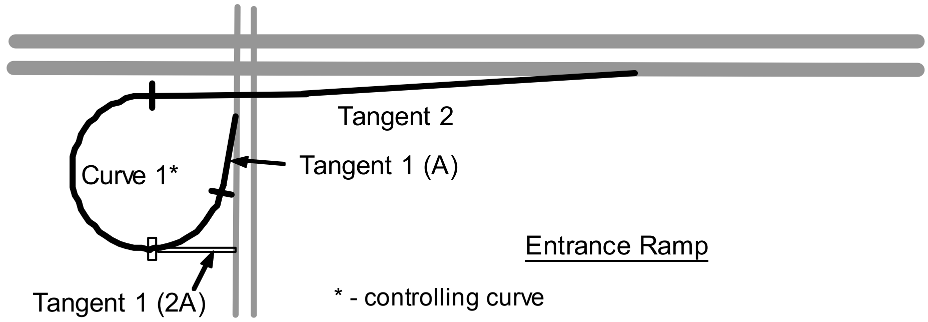


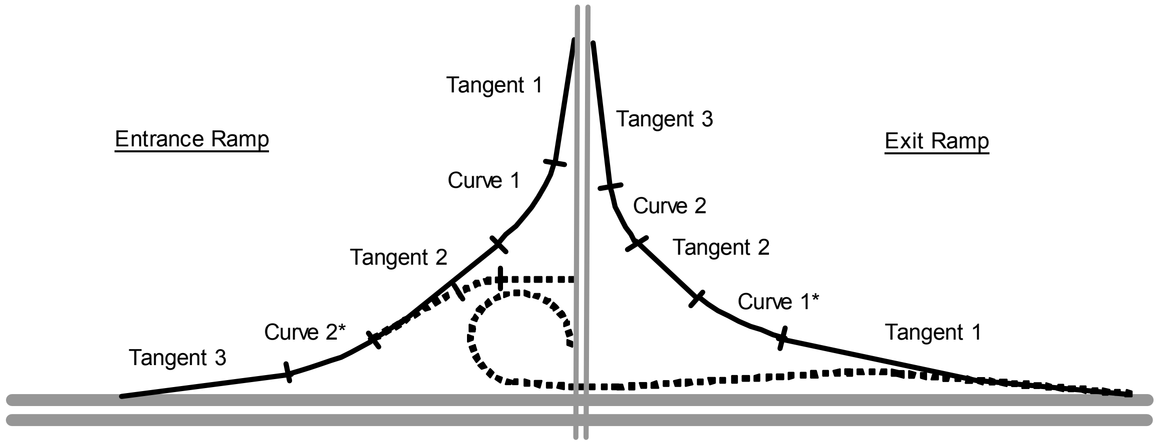
Figure 3-3. Loop Entrance Ramp Segment.

Segments for an outer connection ramp are shown in Figure 3-4. Similar to the diagonal ramp, three tangents and two curves provide the transition from the major-road design speed to the crossroad design speed. For the exit ramp, the deflection angle for Curve 1 should range from 30 to 45 degrees, with larger values associated with a higher design speed on the crossroad. For the entrance ramp, the deflection angle for Curve 1 should range from 45 to 60 degrees, with larger values associated with a higher design speed on the major road. Some variation from these ranges will occur when the major road and crossroad do not intersect at a right angle.

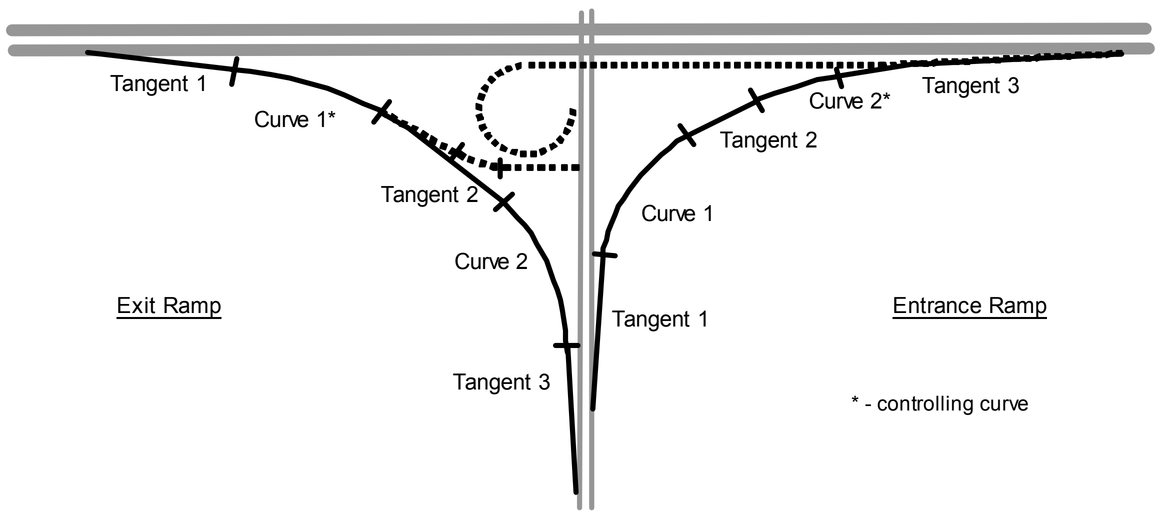
The entrance ramp for the parclo B and the exit ramp for the Parclo A are shown in Figure 3-4 to have a "combined" outer connection and diagonal ramp geometry. Its geometry was previously shown in Figure 2-4. This type of ramp combines the best operational features of the diagonal and outer connector ramps. The left-turn movement is served at the intersection, and the right-turn movement is served by a turning roadway. A disadvantage of this design is that it requires considerable right-of-way and significant distance between the interchange and the nearest downstream intersection on the crossroad. When these disadvantages are significant, a diagonal ramp is often used in isolation to serve both the left-turn and right-turn movements at the crossroad.

Ramp Curve Design Speeds

This section describes rules for assigning a design speed to each of the ramp segments (i.e., curve or tangent) identified in the preceding subsection. The primary goal of the assignment is to provide a uniform, gradual transition between the design speed of the major road and that of the crossroad or ramp terminal. To achieve this goal, a design speed is assigned to each successive curve and tangent pair. The change in design speed among adjacent segments is desirably limited to 5 or 10 mph. Speed changes in this range have been found to yield acceptable levels of deceleration (or acceleration) and are consistent with driver expectancy (*15*).



a. Outer Connection Ramps at Parclo B.



b. Outer Connection Ramps at Parclo A.

Figure 3-4. Outer Connection Ramp Segments.

Assignment of a design speed to each segment begins with establishing the design speed of the "controlling" ramp curve. This curve is identified by an asterisk (*) in Figures 3-1 through 3-4. This design speed is specified in Table 3-20 of the *Roadway Design Manual* (*9*). The speeds specified in this manual are listed in Table 3-5. Column 1 of this table has been modified to show the relationship between ramp configuration and practical design speed values. This relationship is based on guidance provided on page 829 of the *Green Book* (*10*).

Table 3-5. Design Speed for Controlling Ramp Curve.

Ramp Configuration	Major-Road Design Speed, mph										
	30	35	40	45	50	55	60	65	70	75	80
	Controlling Curve Design Speed, mph										
Outer Connection[1]	25	30	35	40	45	48	50	55	60	65	70
Diagonal	20	25	30	33	35	40	45	45	50	55	60
Loop[2]	20	25	25	25	25	25	25	25	25	25	25

Note:

1 - For outer connection ramps, the design speed of Curve 2 for the exit ramp and Curve 1 for the entrance ramp can also be obtained from this table by using the "outer connection" row and the column corresponding to the crossroad design speed (instead of the major-road design speed).

2 - Design speed of the controlling curve on the loop exit ramp is the same as for the diagonal exit ramp.

For outer connection ramps, the ramp curve nearest to the crossroad (i.e., Curve 2 for the exit ramp and Curve 1 for the entrance ramp) can also be considered as a controlling curve for the purpose of identifying its design speed. The design speed for this curve is also obtained from Table 3-5; however, the column used should correspond to the *crossroad* design speed. For example, consider an outer connection exit ramp merging with a crossroad. If the crossroad has a design speed of 50 mph, Table 3-5 indicates that Curve 2 should have a design speed of 45 mph.

Curve 3 on the loop exit ramp is limited to a maximum design speed of 30 mph. Use of a higher speed would yield radii that need significant right-of-way to construct. Also, these large radii would require that left-turning drivers travel considerable extra distance, which would offset any benefit the larger radii might provide in the form of an increased running speed (*10*).

The design speed of the tangent adjoining the major road (or crossroad) is determined by the manner of its intersection with the major road (or crossroad). If this tangent merges with the major road (or crossroad), then its design speed should be equal to that of the major road (or crossroad). If it diverges from the major road, then its design speed should be 5 to 10 mph lower than that of the major road (or crossroad). A 10-mph reduction is typically used when the major road (or crossroad) design speed is above 45 mph. Larger reductions may be needed in some specific instances. A special case applies for Curve 1 of the Parclo A loop entrance ramp. The design speed for the tangent diverging from the crossroad (preceding Curve 1) should not exceed 35 mph.

If the tangent intersects the crossroad at a right angle and is part of an exit ramp, then its design speed is specified as a "stop" condition. This specification applies only to the determination of the tangent-length-related control values that are defined by speed and described in this chapter. A design speed that is consistent with that of the upstream curve should be used to define other control values for this tangent (e.g., stopping sight distance, vertical curvature, grade, etc.).

The design speed for each of the remaining segments is then determined in a sequential manner proceeding along the ramp from the controlling curve to the crossroad. In sequence, the design speed for each successive segment is determined by decreasing its speed, relative to the previous segment, by an amount that ranges from 5 to 10 mph. A 10-mph reduction is generally reserved for those segments with a design speed larger than 45 mph.

Some segment length controls described in this chapter require specification of a speed at the start and end of the segment. For this purpose, the speed at the end of the segment is considered to be equal to the segment design speed; the speed at the start of the segment is considered to be equal to the design speed of the preceding segment. In this application, "start" and "end" are defined in the direction of travel.

Table 3-6 identifies the recommended design speed for each ramp segment. These speeds are based on the application of the aforementioned rules for establishing segment design speeds. For each combination of ramp type and configuration, segments are listed from top to bottom in the direction of travel.

HORIZONTAL GEOMETRICS

This section discusses selected design elements that compose the horizontal alignment of the interchange ramp. As noted previously, the focus of this section is on the elements that are unique to ramps in non-frontage-road settings and for which guidance is not available in the *Roadway Design Manual* (*9*). The topics discussed include: maximum superelevation rate, minimum radius, and minimum length controls for the individual ramp segments.

Maximum Superelevation Rate

The maximum superelevation rate for ramp curves can be either 6.0 or 8.0 percent. Use of a maximum superelevation rate of 8.0 percent allows for smaller curve radii; however, it also tends to increase the minimum length of the tangents by 5 to 8 percent. This increase is due to the additional length needed for superelevation transition. For this reason, a 6.0 percent maximum superelevation rate is preferred.

The one exception to the preference for a 6.0 percent maximum rate is the use of 8.0 percent for the sharpest curve on the loop ramp (i.e., Curve 3 on the loop exit ramp and Curve 1 on the loop entrance ramp). The higher rate of 8.0 percent is recommended for the sharpest curve on the loop ramp because it enables the use of radii of reasonable size. Use of a smaller maximum rate tends to yield radii that need significant right-of-way to construct and require left-turning drivers to travel considerable extra distance (*10*).

Minimum Radius

The minimum radius associated with maximum superelevation rates of 6.0 and 8.0 percent are listed in Table 3-7.

Table 3-6. Ramp Segment Design Speed.

Ramp Type	Ramp Config-uration	Segment[1]	Major-Road Design Speed, mph						
			50	55	60	65	70	75	80
			Ramp Segment Design Speed, mph						
Exit	Diagonal	Tangent 1	40	45	50	55	60	65	70
		Curve 1*	35	40	45	45	50	55	60
		Tangent 2	30	35	40	40	40	45	50
		Curve 2	25	30	35	35	35	40	40
		Tangent 3	stop	stop	stop	stop	stop	stop	stop
	Outer Connection	Tangent 1	45	48	50	55	60	65	70
		Curve 1*	45	48	50	55	60	65	70
		Tangent 2	Average of design speed for Curves 1 and 2.						
		Curve 2	Use Table 3-5[2].						
		Tangent 3	Use crossroad design speed V_{cr}.						
	Loop	Tangent 1	40	45	50	55	60	65	70
		Curve 1*	35	40	45	45	50	55	60
		Tangent 2	30	35	40	40	40	45	50
		Curve 2	25	30	35	35	35	40	40
		Curve 3	25	25	30	30	30	30	30
		Tangent 3 (Parclo B)	Use crossroad design speed V_{cr}.						
		Tangent 3 (Parclo 2B)	stop	stop	stop	stop	stop	stop	stop
Entrance	Diagonal	Tan. 1 (entry speed = 15 mph)	20	25	30	30	30	35	35
		Curve 1	25	30	35	35	35	40	40
		Tangent 2	30	35	40	40	40	45	50
		Curve 2*	35	40	45	45	50	55	60
		Tangent 3	50	55	60	65	70	75	80
	Outer Connection	Tan. 1 (entry speed = V_{cr})	Same as Curve 1.						
		Curve 1	Use Table 3-5[2].						
		Tangent 2	Average of design speed for Curves 1 and 2.						
		Curve 2*	45	48	50	55	60	65	70
		Tangent 3	50	55	60	65	70	75	80
	Loop	Tan. 1 (A) (entry speed = V_{cr})	35	35	35	35	35	35	35
		Tan. 1 (2A) (entry speed = 15)	20	20	20	20	20	20	20
		Curve 1*	25	25	25	25	25	25	25
		Tangent 2	50	55	60	65	70	75	80

Notes:

1 - Segment locations are listed in the direction of travel. Segment numbers are shown in Figures 3-1 through 3-4. For computing some control values, design speeds listed are defined to occur at the *end* of the segment.

2 - Curve design speed can be obtained from Table 3-5 by using crossroad design speed (instead of major-road design speed) and selecting a speed from the row labeled "outer connection."

* - Controlling curve.

Table 3-7. Minimum Radius by Curve Design Speed.

Curve Design Speed, mph	Minimum Curve Radius, ft	
	6.0% Maximum Superelevation	8.0% Maximum Superelevation
25	185	170
30	275	250
35	380	350
40	510	465
45	660	600
50	835	760
55	1065	965
60	1340	1205
65	1660	1485
70	2050	1820

Tables 3-6 and 3-7 were used to determine the minimum radius for each curve of each ramp type and configuration combination. The resulting radii are listed in Table 3-8. Larger radii can be used to accommodate the unique features of a specific location. If above-minimum radii are used, Table 2-6 or 2-7 in the *Roadway Design Manual* (*9*) should be used to identify the appropriate design superelevation rate.

Table 3-8. Minimum Radius for Ramp Curves.

Ramp Type	Ramp Configuration	Segment [1]	Major-Road Design Speed, mph						
			50	55	60	65	70	75	80
			Minimum Curve Radius [2], ft						
Exit	Diagonal	Curve 1	380	510	660	660	835	1065	1340
		Curve 2	185	275	380	380	380	510	510
	Outer Connection	Curve 1	660	765	835	1065	1340	1660	2050
		Curve 2	Varies, depending on design speed selected for Curve 2.						
	Loop [3]	Curve 1	380	510	660	660	835	1065	1340
		Curve 2	185	275	380	380	380	510	510
		Curve 3	170	170	250	250	250	250	250
Entrance	Diagonal	Curve 1	185	275	380	380	380	510	510
		Curve 2	380	510	660	660	835	1065	1340
	Outer Connection	Curve 1	Varies, depending on design speed selected for Curve 1.						
		Curve 2	660	765	835	1065	1340	1660	2050
	Loop [3]	Curve 1	170	170	170	170	170	170	170

Notes:
1 - Segment locations are listed in the direction of travel. Segment numbers are shown in Figures 3-1 through 3-4.
2 - Radius for each curve is based on the curve design speed listed in Table 3-6.
3 - Curve 3 of the loop exit ramp and Curve 1 of the loop entrance ramp are based on a maximum superelevation rate of 8.0 percent; all other curves are based on a maximum superelevation rate of 6.0 percent.

Ramp Length Based on Vertical Alignment

Ramp length is dictated by many controls including those related to: vertical alignment, speed change, and storage requirements. With regard to vertical alignment, ramp length is dictated by stopping sight distance, vertical curve length, grade, and the elevation change needed to vertically separate the major road from the crossroad. In this section, the referenced ramp length is measured from the exit (or entrance) gore on the major road to either: (1) the ramp terminal (if stop-controlled), or (2) the entrance (or exit) gore on the crossroad. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other ramp configurations.

Figure 3-5 illustrates the minimum length dictated by vertical curvature for a 22-ft elevation difference between the major road and crossroad. The circled ends of the lines indicate the point below which the corresponding ramp grade is not feasible. Figure 3-5a applies when the ramp profile undergoes the full elevation change and the major road remains at grade. Figure 3-5b applies when the major road undergoes the elevation change and the ramps remain at grade.

The design speed listed in Figure 3-5a is that assigned to the controlling horizontal curve, as listed in Table 3-5. The second vertical curve on the ramp is assumed to have a design speed that is 5 to 10 mph lower than that of the controlling curve (5 mph is used when the controlling curve design speed is 45 mph or less; 10 mph otherwise). The design speed listed in Figure 3-5b is that of the major road.

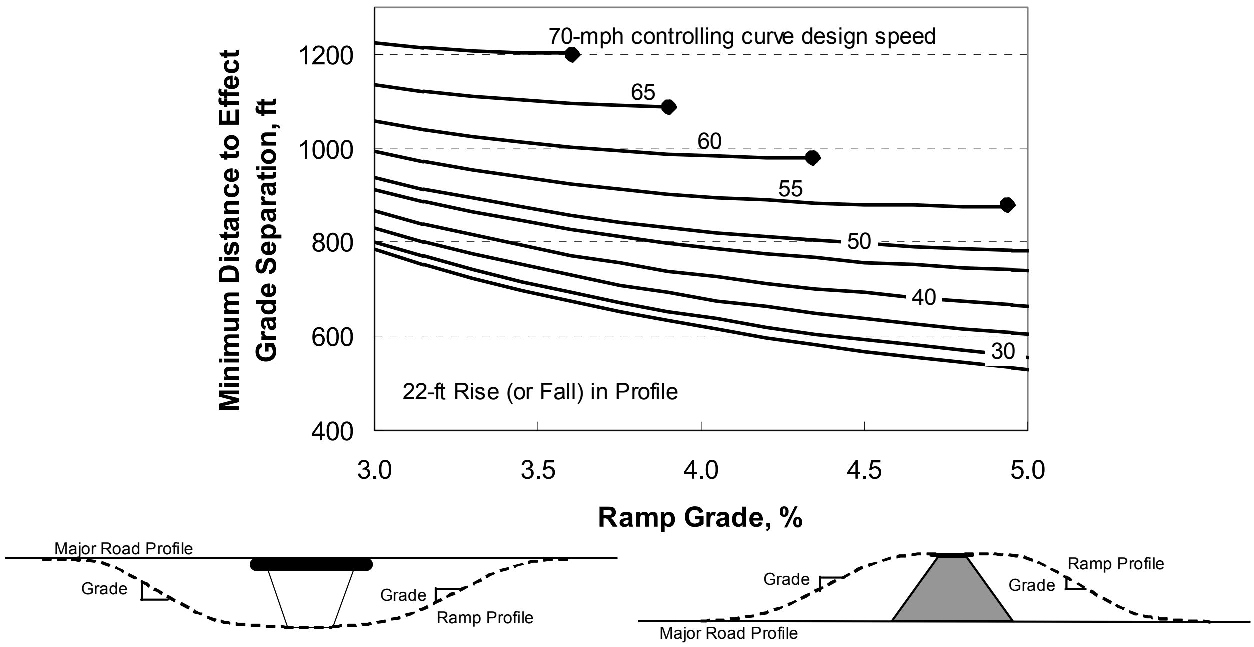
Comparison of Figures 3-5a and 3-5b indicate that shorter ramps may be realized when the ramps undergo the necessary elevation change. This trend is due to the lower design speed of the ramp relative to that of the major road.

Ramp grades of 4.0 percent or less are preferable (*9*). However, grades up to 5.0 percent do not unduly interfere with truck operation and may be used where appropriate for topographic conditions (*10*). Downgrades on the sharpest curve on the loop ramp (i.e., Curve 1 on the parclo A; Curve 3 of the parclo B) should be limited to a maximum of 4.0 percent (*10*).

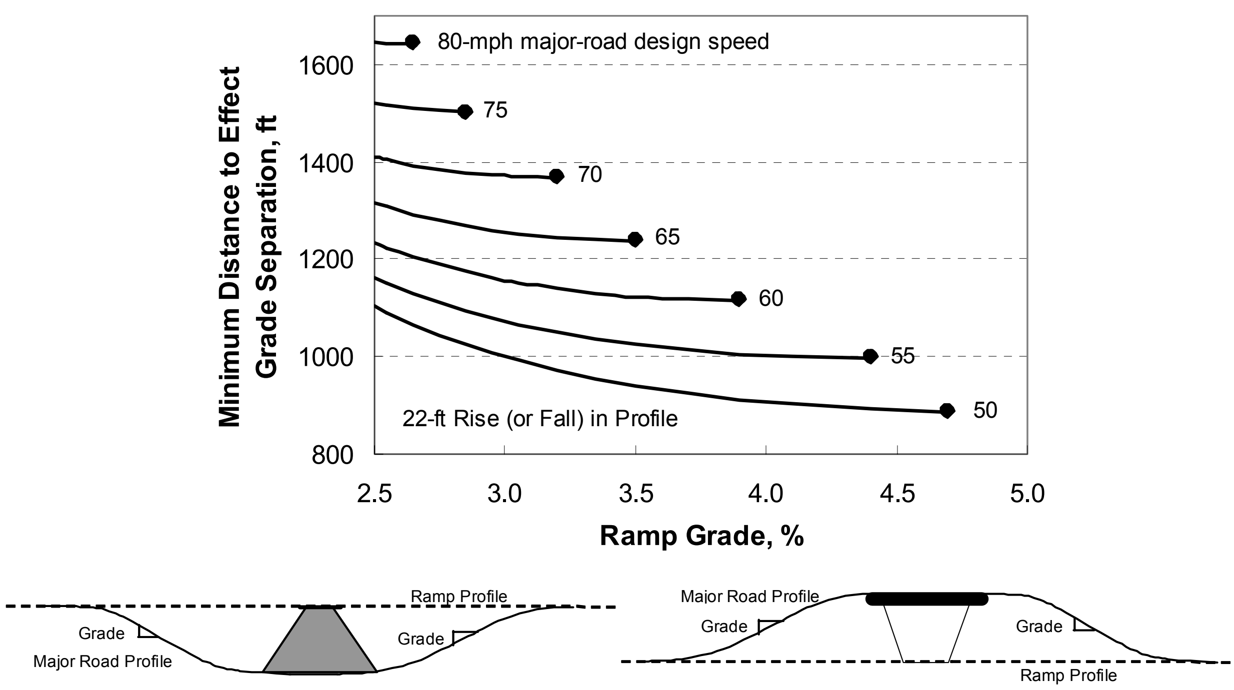
Ramp Length Based on Speed Change and Storage

This section describes design controls defining the minimum length of specific portions of the ramp. These controls include: (1) the minimum length required to effect the change in design speed between the major road and ramp junction, and (2) the length needed for queue storage. Considerations of queue storage are appropriate for exit ramps that terminate at a stop- or signal-controlled junction. They are also appropriate for entrance ramps that have a ramp meter.

In this section, the referenced ramp length is measured from the point on the ramp at which a full-width traffic lane is first developed to either: (1) the ramp terminal (if stop-controlled), or (2) the point on the ramp at which the full-width traffic lane ends. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other ramp configurations.



a. Ramp Profile Undergoes Elevation Change.



b. Major-Road Profile Undergoes Elevation Change.

Figure 3-5. Minimum Ramp Length to Effect Grade Separation.

Speed Change and Storage Segments

One minimum ramp length control is based on the sum of the distance needed for speed change and that needed for queue storage. The relationships between ramp length, speed-change length, and storage length are illustrated in Figure 3-6 for diagonal ramps. These relationships also apply to the other ramp configurations.

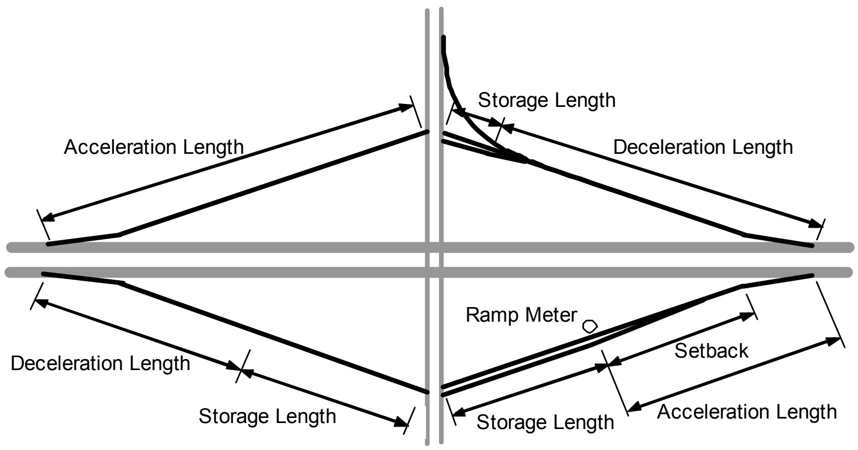


Figure 3-6. Ramp Length Components.

The minimum ramp length for speed change and storage is dependent on ramp type and whether traffic control conditions dictate the need for a storage length component. For exit ramps terminating in a stop condition, the minimum length is based on the distance needed to decelerate from the major-road design speed to a stop condition plus that needed for queue storage. For exit ramps terminating in a merge area, the minimum ramp length is based only on the distance needed to decelerate from the major-road design speed. For unmetered entrance ramps, the minimum ramp length is based only on the distance needed to accelerate from the ramp terminal design speed to that of the major road. For metered entrance ramps, the minimum ramp length is based on the distance needed for storage in advance of the meter plus that needed to accelerate from a stop condition to the design speed of the major road.

Component Lengths

This section provides guidance for determining the minimum ramp length needed for speed change and storage. The discussion focuses on the individual components that combine to define this minimum length. Design controls specifying the minimum lengths for the components include: deceleration, storage, acceleration, and ramp meter setback. Each of these controls is discussed in the remainder of this subsection.

Deceleration Length. The length of ramp needed to decelerate the vehicle from an initial design speed to a final design speed can be obtained from Table 3-9. Most of the lengths shown in this table were obtained from Figure 3-36 of the *Roadway Design Manual* (*9*). Lengths for additional speed combinations in Table 3-9 were determined by extrapolating the trends in the original figure.

Table 3-9. Minimum Length for Deceleration.

Initial Design Speed, mph	Final Design Speed, mph													
	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
	Minimum Length for Deceleration, ft													
20	150	80	--	--	--	--	--	--	--	--	--	--	--	--
25	190	150	100	--	--	--	--	--	--	--	--	--	--	--
30	235	200	170	140	--	--	--	--	--	--	--	--	--	--
35	280	250	210	185	150	--	--	--	--	--	--	--	--	--
40	320	295	265	235	185	155	--	--	--	--	--	--	--	--
45	385	350	325	295	250	220	140	--	--	--	--	--	--	--
50	435	405	385	355	315	285	225	175	--	--	--	--	--	--
55	480	455	440	410	380	350	285	235	140	--	--	--	--	--
60	530	500	480	460	430	405	350	300	240	130	--	--	--	--
65	570	540	520	500	470	440	390	340	280	220	120	--	--	--
70	615	590	570	550	520	490	440	390	340	280	200	110	--	--
75	660	635	620	600	575	535	490	440	390	330	260	190	100	--
80	720	690	670	640	610	570	530	480	430	370	310	240	170	90

The deceleration lengths in Table 3-9 can be adjusted to account for the effect of grade. The appropriate adjustment factor can be obtained from Figure 3-7. The length obtained from Table 3-9 would be multiplied by this factor to compute a deceleration length adjusted for ramp grade. The trends in Figure 3-7 were derived from the adjustment factors provided in Table 3-14 of the *Roadway Design Manual* (*9*).

Exit Ramp Storage Length. The length of ramp needed for queue storage can be calculated using the following equation:

$$L_{\min,q} = f\, S\, \frac{Q\ r}{3600\ n} \qquad \textbf{(13)}$$

where,

$L_{min,\,q}$ = minimum length of roadway needed to store queued vehicles, ft;

f = adjustment factor to provide for storage of all left-turn vehicles on most cycles (= 2.0);

S = average distance between two queued vehicles (= 25, 30, 35, or 40 ft for truck percentage ranges of 0 to 4, 5 to 9, 10 to 14, or 15 to 19 percent), ft;
Q = ramp design hour left-turn volume, veh/h;
r = time during which vehicles queue (unsignalized: 120 s; signalized: 0.75 C), s;
C = signal cycle length, s; and
n = number of lanes available for queue storage.

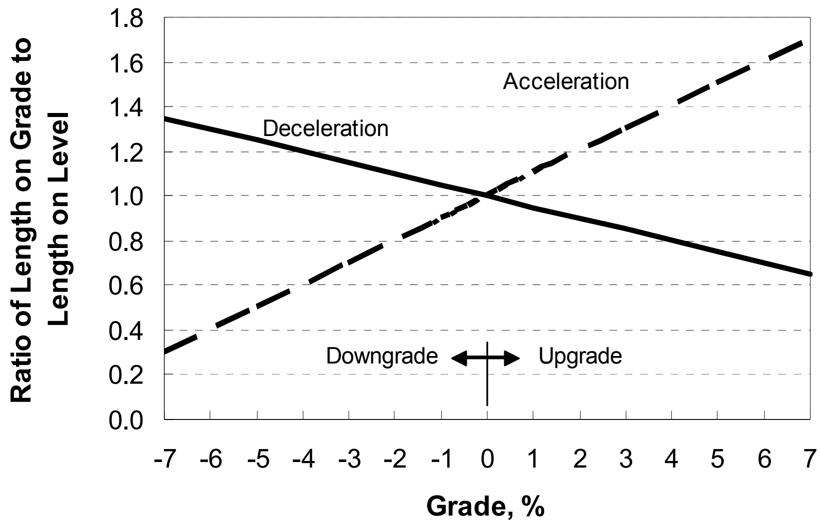


Figure 3-7. Grade Adjustment Factors for Deceleration and Acceleration Lengths.

Equation 13 estimates the maximum number of vehicles likely to require storage on the ramp. It accounts for random fluctuations in arrivals such that the computed length is rarely exceeded; as such this length is appropriate for design. As an alternative to Equation 13, the maximum-back-of-queue statistic obtained from an acceptable software traffic model can also be used for design.

Equation 13 applies to both unsignalized and signalized ramp terminals. For unsignalized intersections, Table 3-3 in the *Roadway Design Manual* (*9*) recommends the use of 120 s as the "time during which vehicles queue" r. For signalized intersections, the discussion in the *Roadway Design Manual* (*9*) recommends a value of r equal to the signal cycle length. However, this recommendation is very conservative because vehicles store at a signalized intersection primarily when the signal indication is red. It is even more conservative when applied to interchange ramp terminals because they tend to have very long cycle lengths. For these reasons, it is suggested that the value of r in Equation 13 should be set equal to the red duration in the ramp signal cycle. For most interchanges, r can be estimated as being about 75 percent of the cycle length.

The storage lengths obtained from Equation 13 for a range of left-turn volumes, left-turn lanes, and ramp terminal control conditions are listed in Table 3-10. The lengths listed in this table reflect a cycle length of 120 s and a truck percentage in the range of 5 to 9 percent.

Table 3-10. Storage Lengths for Signalized and Unsignalized Exit Ramp Terminals.

Signalized Terminals [1]						Unsignalized Terminals [2]	
1 Left-Turn Lane		2 Left-Turn Lanes		3 Left-Turn Lanes		1 Left-Turn Lane	
Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft	Ramp Left-Turn Volume, veh/h	Storage Length, ft
100	150	500	375	900	450	50	100
150	225	550	415	950	475	100	200
200	300	600	450	1000	500	150	300
250	375	650	490	1050	525	200	400
300	450	700	525	1100	550	250	500
350	525	750	565	1150	575	300	600
400	600	800	600	1200	600	350	700

Notes:
1 - Lengths are based on an assumed 5 to 9 percent trucks, a 120-s signal cycle, and a 90-s red duration for the ramp.
2 - Lengths are based on an assumed 5 to 9 percent trucks.

The trends in Table 3-10 suggest that it is desirable that the ramp design provide 600 ft of queue storage. This storage length would serve left-turn volumes up to 400 veh/h. If left-turn volumes exceed 400 veh/h, a second 600-ft storage lane should be included in the design. Two 600-ft lanes would adequately serve left-turn volumes up to 800 veh/h. If the volume exceeds 800 veh/h, then a third 600-ft storage lane is needed.

The last two columns of Table 3-10 indicate the storage length needed for unsignalized ramp terminals (i.e., where the ramp left-turn is stop-controlled). The trends in these columns indicate it is desirable that the ramp design include 600 ft for queue storage. In this manner, when left-turn volumes reach 300 veh/h, signalization of the ramp terminals is likely to be justified. Once the terminal is signalized, the 600 ft of storage will adequately serve a left-turn volume up to 400 veh/h.

In summary, the approach of providing 600 ft for storage on the ramp (regardless of volume or ramp terminal traffic control) is a desirable practice because it provides flexibility in the design. The ramp can be adapted to increasing traffic volume over its service life by adding additional lanes (instead of increasing ramp length). This approach is also cost-effective at unsignalized terminals because, as traffic increases over time, the need for signalization is likely to be coincident with the need to address ramp queue storage (rather than one need occurring before the other).

Storage Length for Ramp Meter Control. As indicated in Figure 3-6, an entrance ramp controlled by a ramp meter requires a storage area to safely store vehicles between the crossroad ramp terminal and the meter. Guidance for designing an entrance ramp with a ramp meter is provided in *Design Criteria for Ramp Metering* (*16*). The recommended storage lengths in this document were modified to account for the absence of frontage roads. Specifically, this modification included the use of a 150-ft minimum distance to the back of queue instead of the 250 ft recommended for ramps with frontage roads. This distance is adequate to permit traffic entering the ramp to perceive, and safely stop behind, the queue. The recommended minimum storage lengths are summarized in Table 3-11.

Table 3-11. Storage Lengths for Entrance Ramps with Ramp Meter Control.

1-Lane Ramp/Single Release		1-Lane Ramp/Multiple Release		2-Lane Ramp/Single Release	
Ramp Volume, veh/h	**Storage Length, ft**	**Ramp Volume, veh/h**	**Storage Length, ft**	**Ramp Volume, veh/h**	**Storage Length, ft**
200	310	600	555	1000	725
300	395	700	605	1100	755
400	480	800	650	1200	785
500	560	900	690	1300	805
600	640	1000	730	1400	820
700	725	1100	760	1500	830
800	800	1200	785	1600	840

Ramp volumes in excess of meter capacity will cause the meter to occasionally "flush" (i.e., show steady green), which can cause delay to major-road traffic. Hence, the need for single or dual-lane ramp storage should be determined by the capacity of the single-lane ramp meter. The capacity of a single-lane ramp meter with single-vehicle release per cycle is 800 veh/h (*16*). If two or three vehicles are released per cycle, the capacity is 1100 to 1200 veh/h. The capacity of a dual-lane meter is 1600 veh/h (*16*).

In general, it is preferable that all entrance ramps in urban areas include storage length for ramp meter control. This practice will ensure adequate ramp length is available in the event that the ramp is metered at some point during its design life. Based on the trends in Table 3-11, it is desirable that the entrance ramp design include 800 ft for queue storage. Provision of at least 800 ft of storage will allow a single-lane ramp to adequately serve ramp volumes up to 1200 veh/h. If volumes exceed this amount, then a second lane should be added to the ramp. A metered two-lane ramp will serve traffic volumes up to 1600 veh/h.

Acceleration Length. The length of ramp needed to accelerate the vehicle from an initial design speed to a final design speed can be obtained from Table 3-12. Most of the lengths shown in this table were obtained from Figure 3-36 of the *Roadway Design Manual* (*9*). Lengths for additional speed combinations in Table 3-12 were determined by extrapolating the trends in the original figure.

Table 3-12. Minimum Length for Acceleration.

Final Design Speed, mph	Initial Design Speed, mph													
	Stop	15	20	25	30	35	40	45	50	55	60	65	70	75
	Minimum Length for Acceleration, ft													
20	70	10	--	--	--	--	--	--	--	--	--	--	--	--
25	120	60	10	--	--	--	--	--	--	--	--	--	--	--
30	180	140	80	20	--	--	--	--	--	--	--	--	--	--
35	280	220	160	110	20	--	--	--	--	--	--	--	--	--
40	360	300	270	210	120	30	--	--	--	--	--	--	--	--
45	560	490	440	380	280	160	30	--	--	--	--	--	--	--
50	720	660	610	550	450	350	130	30	--	--	--	--	--	--
55	960	900	810	780	670	550	320	150	30	--	--	--	--	--
60	1200	1140	1100	1020	910	800	550	420	180	30	--	--	--	--
65	1410	1350	1310	1220	1120	1000	770	600	370	140	30	--	--	--
70	1620	1560	1520	1420	1350	1230	1000	820	580	370	160	30	--	--
75	1790	1730	1630	1580	1510	1420	1160	1040	780	540	330	90	30	--
80	2000	1920	1860	1790	1690	1580	1360	1180	970	720	510	270	90	30

Table 3-12 can be used to determine the minimum length of ramp needed when a meter is planned. In this application, the acceleration length is provided in addition to the storage length needed for the meter. The initial design speed used with Table 3-12 is the "stop" condition. The final design speed ranges from 50 to 60 mph in correlation with the major-road design speed range of 50 to 80 mph. This final speed reflects a tendency for ramps to be metered only during peak traffic periods when operating speeds in the shoulder lane tend not to exceed 50 mph.

The acceleration lengths in Table 3-12 can be adjusted to account for the effect of grade. The appropriate adjustment factor can be obtained from Figure 3-7. The length obtained from Table 3-12 would be multiplied by this factor to compute an acceleration length adjusted for ramp grade.

Setback For Ramp Meter. To ensure reasonable horizontal clearance between the major road and ramp meter, a nominal setback distance is needed between the ramp meter and ramp gore. The location of this distance is shown in Figure 3-6. Minimum and desirable values for this distance are 250 and 350 ft, respectively. These distances are based on a synthesis of the ramp meter setback distances used by nine state departments of transportation, as reported by Lomax and Fuhs (*17*).

Minimum Ramp Length Based on Speed Change and Storage

The minimum length controls described in the previous subsection can be used to examine their impact on ramp length. Controls addressed in this section include: deceleration, storage, and acceleration. The relationship between the associated lengths was shown previously in Figure 3-6.

The information in this section is based on the assumption that the ramps are on level terrain. Other lengths will be obtained if the ramps are on grade. The objective of this section is to illustrate how the noted design controls can be used together to assess their impact on overall ramp length.

The discussion in this section focuses on the diagonal and 2-quad parclo ramp configurations because the design speed for the ramp segment adjoining the crossroad is known. The other ramp configurations are not addressed because they require specification of a crossroad design speed. Nevertheless, minimum ramp length based on speed change and storage can be determined for these ramps once the crossroad design speed is specified. The minimum ramp lengths for the diagonal and 2-quad parclo ramps are listed in Table 3-13.

Table 3-13. Minimum Ramp Length Based on Speed Change and Storage.

Ramp Type	Ramp Configuration	Component	Major-Road Design Speed, mph						
			50	55	60	65	70	75	80
			Minimum Ramp Length, ft						
Exit	Diagonal, Parclo B (2-quad)	Storage	600	600	600	600	600	600	600
		Deceleration[1]	435	480	530	570	615	660	720
		Total:	1035	1080	1130	1170	1215	1260	1320
Entrance	Diagonal, Parclo A (2-quad)	Acceleration[1]	660	900	1140	1350	1560	1730	1920
		Total:	660	900	1140	1350	1560	1730	1920
	Metered Ramp	Storage	800	800	800	800	800	800	800
		Acceleration[2]	720	830	900	960	1050	1130	1200
		Total:	1520	1630	1700	1760	1850	1930	2000

Notes:

1 - Lengths are based on the major-road design speed and the design speed of the ramp terminal. This latter speed is defined as "stop" condition and 15 mph for the exit and entrance ramps, respectively.

2 - Lengths are based on the assumption that metering occurs during peak traffic periods. The speed during these periods is estimated to be in the range of 50 to 60 mph and increases with major-road design speed.

The total lengths listed in Table 3-13 are measured from the point on the ramp at which a full-width traffic lane is first developed to the ramp terminal.

Ramp Length Based on Design Speed

This section describes controls that specify the minimum length of the individual ramp segments. These controls include: curve travel time, superelevation transition length on tangent, and segment speed change.

Discussion in this section referring to ramp length is based on measurement from the point on the major road at which a full-width traffic lane is provided for diverging (or merging) ramp traffic to either: (1) the ramp terminal (if stop-controlled), or (2) the point on the crossroad where the full-width traffic lane ends for merging (or diverging) ramp traffic. The former point applies to the diagonal and 2-quad parclo ramp configurations; the latter applies to the other configurations.

In some situations, constraints imposed by environmental, cost, or right-of-way considerations result in the ramp being designed to its minimum practical length. In these situations, some of the minimum segment length controls defined in this section are likely to dictate segment length. Rarely, if ever, will all of the segments be at the minimum values specified by these controls. Factors related to the geometry of the ramp alignment, skew angle, and topography will often serve to dictate the length and orientation of the other ramp segments.

Component Lengths

Travel Time Length. Page 234 of the *Green Book* (*10*) indicates that highway curves should have a minimum length equal to 15 times the design speed expressed in miles per hour. This control equates to a minimum curve length of 10-s travel time at the design speed. This length is excessive for ramp curve design; however, research on curve driving behavior indicates that 3.0-s travel time is necessary to accommodate the steering maneuver during curve entry and exit (*18*). This travel time is slightly longer than the 2.0 s used to define the desirable length of a spiral transition curve; it also tends to provide curve lengths that are in balance with the minimum tangent lengths needed for superelevation transition (*10*). For these reasons, the minimum length of ramp curve is defined to equal 3.0-s travel time. This length can be computed using the following equation:

$$L_{\min,t} = 4.4\ V_c \qquad \textbf{(14)}$$

where,
$L_{min,\,t}$ = minimum length of ramp curve based on travel time, ft; and
V_c = curve design speed, mph.

Transition Length. This control often dictates the minimum length of tangent between two ramp curves. It is intended to provide sufficient length for superelevation transition from the exited curve to the entered curve. The following equation can be used to calculate this minimum length:

$$L_{\min,r} = w\ p_r \left(\frac{CS_u}{G_u} + \frac{CS_d}{G_d} \right) \qquad \textbf{(15)}$$

where,
$L_{min,\,r}$ = minimum length of tangent between two curves based on superelevation transition, ft;
w = width of rotated traffic lane (i.e., ramp traveled-way width) (see Table 3-17), ft;
p_r = portion of transition located on the tangent (use 0.67);
CS_u = change in pavement cross slope from upstream curve to tangent, percent;
CS_d = change in pavement cross slope from tangent to downstream curve, percent;

G_u = maximum relative gradient for upstream curve (see Table 3-14), percent; and
G_d = maximum relative gradient for downstream curve (see Table 3-14), percent.

Variables in Equation 15 indicate that the minimum length of tangent depends on the width of the ramp traveled-way, the design superelevation rate, and the maximum relative gradient. Desirable traveled-way widths for ramps are the subject of discussion in a subsequent section. Equation 15 is formulated for application to tangents between two curves. It can be used for tangents at the beginning and end of the ramp if one of the "change in cross slope" *CS* terms is deleted.

The design superelevation rate needed for Equation 15 can be obtained from Table 2-6 or 2-7 in the *Roadway Design Manual* (*9*). The maximum relative gradient is dependent on curve design speed. Values of this control are listed in Table 3-14.

Table 3-14. Maximum Relative Gradient.

	Curve Design Speed, mph											
	25	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**65**	**70**	**75**	**80**
Max. Gradient, %	0.70	0.66	0.62	0.58	0.54	0.50	0.47	0.45	0.43	0.40	0.38	0.35

Deceleration Length. As noted previously in the discussion associated with Table 3-6, each ramp segment is associated with a different design speed. Design speeds typically change from 5 to 10 mph between adjacent segments. The length of each segment must be sufficient to allow for comfortable deceleration in conjunction with the change in design speed.

The minimum length of ramp segment needed for deceleration can be obtained from Table 3-9. The "initial" speed is defined as the design speed of the preceding segment. The "final" speed is the design speed of the subject segment. These speeds are identified in Table 3-6 for the various ramp configurations. The lengths in Table 3-9 can be adjusted to account for the effect of grade by using Figure 3-7.

Acceleration Length. The minimum length of ramp segment for acceleration can be obtained from Table 3-12. The "initial" and "final" speeds are defined in the same manner as in the preceding paragraph. The guidance regarding adjustment for ramp grade is also the same.

Balanced Segment Length. Considerations of safety and aesthetics justify the need for balance in the length of the central tangent and its adjacent curves on the outer connection ramp (or any other ramp where two successive curves deflect in the same direction). A tangent that is significantly shorter than that of the adjacent curves results in a "broken-back" arrangement of curves. Such an arrangement is contrary to driver expectancy and can result in an alignment that is not pleasing in appearance. To avoid these complications, Curves 1 and 2 of the outer connection ramp should have about the same length. Moreover, the length of Tangent 2 should equal or exceed the length of Curve 1 (or Curve 2).

Example Application of Segment Design Speed Controls

The minimum length controls described in the previous subsection can be used to define the minimum length of each ramp segment based on its design speed. Controls that dictate these minimums include: curve travel time, superelevation transition length on tangent, and segment speed change. This section describes the application of these controls to the diagonal entrance and exit ramps. The application to the other ramp configurations is identical; however, a crossroad speed is needed to complete the analysis.

The minimum segment lengths for the diagonal exit and entrance ramps are listed in Tables 3-15 and 3-16, respectively. The minimum length shown for each segment represents the larger of the two controls considered. It is unlikely that all the minimum lengths listed would control for a given ramp due to other geometric and topographic factors that tend to dictate ramp alignment. Hence, the values listed in these tables are offered only to illustrate the application of the controls described in the previous section. The total length shown at the bottom of the tables should not be taken as a control on overall ramp length.

Loop Exit Ramp Offset

The loop exit ramp shown in Figure 3-2 has a slight reverse curvature to increase driver awareness and promote a safe reduction in speed prior to the sharp curve at the end of the ramp. The deflection in the ramp alignment should be sufficient to ensure that drivers follow the curved alignment and discourage them from traveling in a straight line through Curves 1 and 2. This can be accomplished by ensuring that Curve 1 is laterally offset from the major road a distance of 40 ft (measured between the nearest edge-of-shoulder for both roadways).

CROSS SECTION

This section discusses issues related to the design of the cross section of the ramp proper. The ramp design elements (and associated controls) that are unique to interchanges in non-frontage-road settings are addressed. These elements include the width of the ramp traveled-way and the number of lanes on the ramp. Design decisions regarding the cross section of the ramp terminal approach are discussed in the next section.

Ramp Traveled-Way Width

Ramp traveled-way widths should be based on ramp curvature, number of lanes provided, and the portion of trucks in the traffic stream to accommodate truck off-tracking. Desirable traveled-way widths for ramps with curbs or shoulders are provided in Table 3-17. The values shown in the table for the "shoulder" edge treatment are based on left and right shoulder widths of 2.0 and 6.0 ft, respectively. Exhibit 10-67 of the *Green Book* (*10*) provides guidance for determining traveled-way width when the sum of the left and right shoulder widths is less than 8.0 ft.

Table 3-15. Application of Segment Controls to Diagonal Exit Ramp.

Segment[1]	Control	Major-Road Design Speed, mph						
		50	55	60	65	70	75	80
		Segment Design Control Values[2]						
Tangent 1	Design Speed, mph:	40	45	50	55	60	65	70
	Max. Gradient (downstream), %:	0.62	0.58	0.54	0.54	0.50	0.47	0.45
	Transition Length, ft:	61	65	69	69	75	80	83
	Deceleration Length, ft:	225	235	240	220	200	190	170
	Minimum Length, ft:	225	235	240	220	200	190	170
Curve 1	Design Speed, mph:	35	40	45	45	50	55	60
	Travel Time Length, ft:	154	176	198	198	221	243	265
	Deceleration Length, ft:	155	140	175	235	240	220	200
	Minimum Length, ft:	155	176	198	235	240	243	265
Tangent 2	Design Speed, mph:	30	35	40	40	40	45	50
	Max. Gradient (upstream), %:	0.62	0.58	0.54	0.54	0.50	0.47	0.45
	Max. Gradient (downstream), %:	0.70	0.66	0.62	0.62	0.62	0.58	0.58
	Transition Length, ft:	171	182	195	195	203	217	222
	Deceleration Length, ft:	150	155	140	140	225	235	240
	Minimum Length, ft:	171	182	195	195	225	235	240
Curve 2	Design Speed, mph:	25	30	35	35	35	40	40
	Travel Time Length, ft:	110	132	154	154	154	176	176
	Deceleration Length, ft:	140	150	155	155	155	140	225
	Minimum Length, ft:	140	150	155	155	155	176	225
Tangent 3	Design Speed, mph:	stop	stop	stop	stop	stop	stop	stop
	Max. Gradient (upstream), %:	0.70	0.66	0.62	0.62	0.62	0.58	0.58
	Transition Length, ft:	107	114	121	121	121	129	129
	Deceleration Length, ft:	190	235	280	280	280	320	320
	Minimum Length, ft:	190	235	280	280	280	320	320
Grand Total Length, ft:		881	979	1068	1085	1100	1164	1220

Notes:

1 - Tangent and curve segment locations listed in direction of travel and are shown in Figure 3-1.

2 - Minimum lengths are based indirectly on the corresponding segment design speeds listed in Table 3-6. Assumptions include: ramps on level terrain, maximum superelevation rate of 6.0 percent, single-lane ramp, left shoulder of 2.0 ft, right shoulder of 6.0 ft, tangent cross slope of 2.0 percent, and traveled-way width of 14 ft.

Table 3-16. Application of Segment Controls to Diagonal Entrance Ramp.

Segment[1]	Control	Major-Road Design Speed, mph						
		50	55	60	65	70	75	80
		Segment Design Control Values[2]						
Tangent 1	Design Speed, mph:	20	25	30	30	30	35	35
	Max. Gradient (downstream), %:	0.70	0.66	0.62	0.62	0.62	0.58	0.58
	Transition Length, ft:	107	114	121	121	121	129	129
	Acceleration Length, ft:	10	60	140	140	140	220	220
	Minimum Length, ft:	107	114	140	140	140	220	220
Curve 1	Design Speed, mph:	25	30	35	35	35	40	40
	Travel Time Length, ft:	110	132	154	154	154	176	176
	Acceleration Length, ft:	10	20	20	20	20	30	30
	Minimum Length, ft:	110	132	154	154	154	176	176
Tangent 2	Design Speed, mph:	30	35	40	40	40	45	50
	Max. Gradient (upstream), %:	0.70	0.66	0.62	0.62	0.62	0.58	0.58
	Max. Gradient (downstream), %:	0.62	0.58	0.54	0.54	0.50	0.47	0.45
	Transition Length, ft:	171	182	195	195	203	217	222
	Acceleration Length, ft:	20	20	30	30	30	30	130
	Minimum Length, ft:	171	182	195	195	203	217	222
Curve 2	Design Speed, mph:	35	40	45	45	50	55	60
	Travel Time Length, ft:	154	176	198	198	221	243	265
	Acceleration Length, ft:	20	30	30	30	130	150	180
	Minimum Length, ft:	154	176	198	198	221	243	265
Tangent 3	Design Speed, mph:	50	55	60	65	70	75	80
	Max. Gradient (upstream), %:	0.62	0.58	0.54	0.54	0.50	0.47	0.45
	Transition Length, ft:	61	65	69	69	75	80	83
	Acceleration Length, ft:	350	320	420	600	580	540	510
	Minimum Length, ft:	350	320	420	600	580	540	510
Grand Total Length, ft:		893	925	1108	1288	1298	1396	1393

Notes:

1 - Tangent and curve segment locations listed in direction of travel and are shown in Figure 3-1.

2 - Minimum lengths are based indirectly on the corresponding segment design speeds listed in Table 3-6. Assumptions include: ramps on level terrain, maximum superelevation rate of 6.0 percent, single-lane ramp, left shoulder of 2.0 ft, right shoulder of 6.0 ft, and traveled-way width of 14 ft, tangent cross slope of 2.0 percent, and entry speed to Tangent 1 of 15 mph.

The traveled-way widths in Table 3-17 are sensitive to the type of edge treatment provided along the ramp. In general, ramps with shoulders on both the left and right sides require the least traveled-way width because the shoulder can be used for occasional off-tracking by the largest trucks. In contrast, ramps with vertical curbs on both sides require the widest ramp traveled-way.

Table 3-17. Traveled-Way Width for Ramps.

Edge Treatment	Curve Radius, ft	Single-Lane Ramp Truck Percentage			Dual-Lane Ramp Truck Percentage		
		< 5%	5 to 10%	>10%	< 5%	5 to 10%	>10%
		Traveled-Way Width [1], ft					
Mountable Curb - both sides	150	18	21	23	26	29	32
	200	17	20	22	26	28	30
	300	17	20	22	25	28	29
	400	17	19	21	25	27	28
	500	17	19	21	25	27	28
	Tangent	17	18	20	24	26	26
Vertical Curb - one side [2]	150	18	21	23	27	30	33
	200	17	20	22	27	29	31
	300	17	20	22	26	29	30
	400	17	19	21	26	28	29
	500	17	19	21	26	28	29
	Tangent	17	18	20	25	27	27
Shoulder	150	14	15	17	24	27	30
	200	13	15	16	24	26	28
	300	13	15	15	23	26	27
	400	13	15	15	23	25	26
	500	12	15	15	23	25	26
	Tangent	12	14	14	22	24	24

Notes:
1 - Widths from Reference *10* (Exhibit 10-67) and based on left and right shoulder widths of 2.0 and 6.0 ft, respectively.
2 - For ramps with vertical curb on *both* sides, add 1.0 ft to the widths shown.

Number of Lanes

The capacity of the ramp terminal or the merge/diverge point often limits ramp volume to values below that of the capacity of a single traffic lane. For this reason, a single-lane ramp cross section should be adequate for most service interchanges. However, a dual-lane ramp proper may be justified if any of the following conditions are expected:

- Design hour ramp volume exceeds the practical capacity of a single traffic lane.
- Ramp is longer than 1400 ft, in which case a two-lane ramp would allow opportunities to pass slower vehicles.
- Ramp is located on a steep upgrade such that a two-lane ramp would allow opportunities to pass vehicles slowed by the grade.
- Ramp has a long, sharp curve such that a two-lane ramp would provide additional accommodation of off-tracking by long vehicles.

- For entrance ramps, design hour ramp volume exceeds 800 veh/h and the ramp will be metered.

For purposes of evaluating the first bullet, the practical capacity for a single-lane diagonal or outer connection ramp is 1550 veh/h (for dual-lane ramps it is 2950 veh/h) (*19*, Chapter 9). The practical capacity for a loop ramp is 1200 veh/h (for dual-lane loop ramps it is 2150 veh/h) (*19*). The ramp length identified in the second bullet is measured from the exit (or entrance) gore on the major road to either: (1) the ramp terminal (if signal or stop-controlled), or (2) the entrance (or exit) gore on the crossroad.

RAMP TERMINAL DESIGN

Traffic Control

The traffic control mode used to regulate traffic at the ramp terminals has a significant impact on traffic flow along the crossroad and on the extent of queue growth on the exit ramps. The control mode used for the left-turn movement may not be the same as that used to control the right-turn movements. Possible combinations of control mode are listed in Table 3-18.

Table 3-18. Exit-Ramp Traffic Control Combinations.

Exit-Ramp Left-Turn Traffic Control Modes	Exit-Ramp Right-Turn Traffic Control Modes [1]			
	Signal	Stop	Yield	Merge [2]
Signal	✔	not common	✔	✔
Stop	not common	✔	✔	✔

Notes:
1 - Common control mode combinations are indicated by check (✔).
2 - Free (uncontrolled) right-turn lane with an added lane extending beyond the end of the channelizing island and along the crossroad requiring ramp vehicles to merge with crossroad vehicles.

A decision to use signal control at the ramp terminal should be based on an evaluation of the signal warrants provided in Part 4 of the *Manual on Uniform Traffic Control Devices* (*20*) and the findings from a capacity analysis. Other criteria may also be considered in the decision of whether to use signal control. In fact, signal control may be helpful under the following conditions:

- to increase ramp capacity and, thereby, prevent spillback from the ramp onto the major road,
- whenever dual left-turn or right-turn lanes are dictated by traffic demand, and
- when sight distance to ramp drivers is restricted along the crossroad.

Both interchange ramp terminals should use the same traffic control mode to regulate the left-turn movements (e.g., both signalized or both unsignalized).

The free-right-turn lane associated with the "merge" design tends to extend a considerable length along the crossroad and can induce intense weaving activity on the crossroad if the adjacent downstream intersection is relatively close to the ramp terminal. For this reason, the "merge" traffic control mode may be best-suited to locations where the distance to the adjacent downstream intersection will exceed 1320 ft for the design life of the interchange.

Traffic control for pedestrians at signalized interchanges often consists of crosswalks across the ramp approach leg, ramp departure leg, and external crossroad leg. A crosswalk across the internal crossroad leg is typically not provided due to complications associated with the ramp terminal signal phasing.

Intersection Skew Angle

Ramp terminal design for non-frontage-road settings can include a discontinuous alignment through the ramp/crossroad junction. This discontinuity allows each ramp junction leg to be skewed in a direction toward the major road, thereby minimizing the curvature on the ramp proper. Figure 3-8 illustrates the discontinuous alignment of the approach and departure legs at a ramp/crossroad junction. The radius, throat width, and taper rate shown in Figure 3-8 are the subject of discussion in the next section.

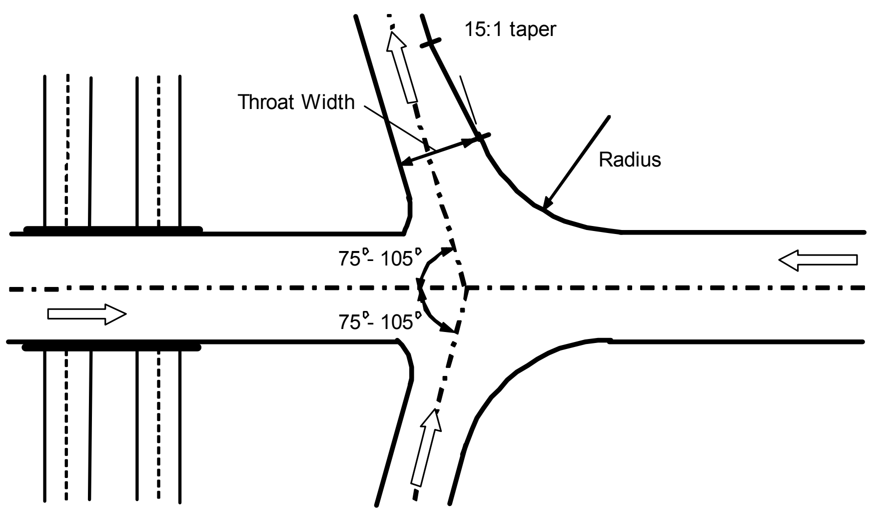


Figure 3-8. Discontinuous Ramp Alignment at Ramp/Crossroad Junction.

Desirably, the alignment of the ramp would be designed such that skew is avoided at the ramp terminal. However, if conditions dictate the use of skew, it is desirable that the skew angle fall within the range of 75 to 105 degrees. Angles larger or smaller than this amount tend to limit the visibility of ramp drivers, increase pedestrian crossing distances, and increase the exposure time for left-turn drivers. In some circumstances, angles in the range of 60 to 120 degrees are acceptable.

Departure Leg Design

The departure leg of the entrance ramp should accommodate the swept path of the design vehicle as it negotiates a left turn or a right turn from the crossroad. The traveled-way within the throat of the departure leg should be sufficiently wide as to allow the design vehicle to turn from the crossroad and enter the ramp without, at any point, encroaching on an adjacent lane, shoulder, or curb. This need can be served by use of a simple corner radius with adequate throat width. It can also be served by use of a three-centered compound curve for the corner radius. Typical three-centered curve designs are provided in Chapter 9 of the *Green Book* (*10*).

If a simple corner radius design is desired, the throat width needed for various radius and design vehicle combinations is provided in Table 3-19. Interpolation is appropriate for intermediate skew angles and radii.

Table 3-19. Ramp Departure Leg Throat Width.

Angle of Intersection, degrees	Corner Radius [1], ft	Throat Width for Selected Design Vehicles [2], ft					
		SU	BUS	WB-40	WB-50	WB-62	WB-67
60	25	17	24	21	28	--	--
	30	16	23	19	25	--	--
	40	14	22	17	22	30	37
	50	14	20	15	20	27	34
	60	14	18	14	18	25	31
90	25	19	30	23	32	--	--
	30	17	25	19	29	--	--
	40	14	22	17	22	39	39
	50	14	19	14	19	33	33
	60	14	16	14	16	29	29
120	25	21	32	24	36	--	--
	30	17	26	19	30	--	--
	40	14	19	17	22	26	30
	50	14	16	14	18	22	25
	60	14	14	14	15	18	21

Notes:
1 - Radii shown represent a simple curve radius design for the edge of traveled way.
2 - Widths from Reference *10* (Exhibit 9-31).
"--" - not applicable.

The throat width identified in Table 3-19 should be provided at the point where the simple radius ends and the tangent portion of the entrance ramp begins. The transition back to the nominal traveled-way width of the ramp, using a 15:1 taper, should also begin at this point. The layout of these design elements is shown in Figure 3-8.

Approach Leg Design

The number of lanes provided on an exit-ramp terminal approach should reflect consideration of the traffic control mode, turn volumes, and crossroad through volume. The guidance provided in a previous section on storage length (see discussion associated with Table 3-10) should be used to determine the minimum number of lanes needed on the ramp terminal approach and their length. Capacity analysis techniques, such as given in the *Highway Capacity Manual* (*21*), can also be used to make these determinations.

As a minimum, it is preferable to provide at least one approach lane for each traffic movement served at the ramp terminal. This treatment permits the use of separate traffic control modes to regulate each movement and should minimize delays and queues to ramp traffic.

Design to Discourage Wrong-Way Maneuvers

A problem inherent to interchanges is the potential for wrong-way entry into an exit ramp. Several techniques have been used to discourage these maneuvers. One technique is to use a sharp corner radius on the inside of the turn movements that, if completed, would result in a wrong-way maneuver. This technique is illustrated at two locations in Figure 3-9. One location is at the intersection of the left-edge of the exit ramp approach and the right-edge of the crossroad approach. It should discourage improper right turns into the exit ramp. A second location is at the median nose on the external crossroad approach. The sharp radius at this location should discourage improper left turns into the exit ramp.

Another technique to discourage wrong-way maneuvers is to use island channelization within the intersection. If used, this channelization should not be overly complex nor should it obstruct the ramp-to-ramp through traffic movement, if it exists. Although this movement typically has negligible traffic volume, its accommodation in the ramp terminal design is important because it can provide essential capacity during incidents or maintenance activities on the major road.

Some parclo A (2-quad) and parclo B (2-quad) designs have experienced wrong-way maneuvers onto the exit ramps. The potential for this maneuver exists because the ramp junction approach and departure legs are located adjacent to one another on the same side of the crossroad. Separation of these two ramp legs using a median of nominal width can provide for the development of a semicircular crossroad median nose that shadows the exit ramp approach and discourages wrong-way entry via a left turn into the exit ramp.

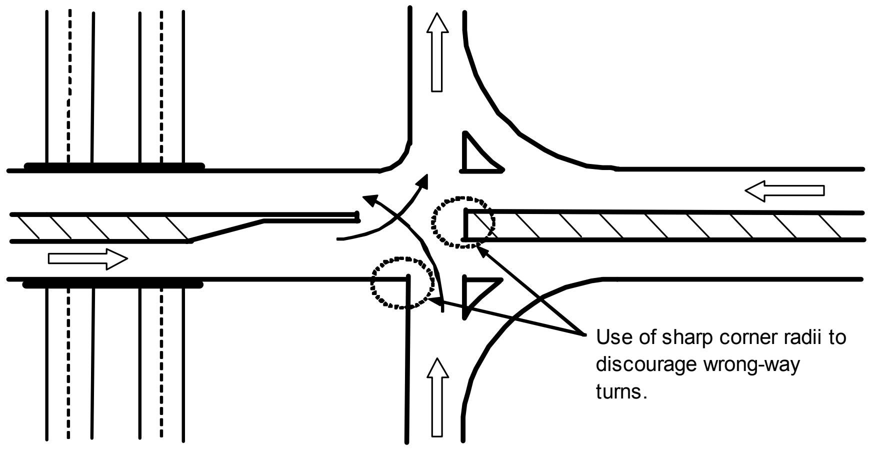


Figure 3-9. Designs to Discourage Wrong-Way Maneuvers.

Access Control on Crossroad

The control of access along the crossroad is essential to the safe and efficient operation of the interchange. The importance of access control is heightened when frontage roads are not provided because of the inherent increase in turning traffic and the focus on development of properties adjacent to the crossroad. Inadequate access control in the vicinity of the interchange can create operational problems on the crossroad that may propagate to the ramps, causing spillback onto the major road. To ensure efficient interchange operation, access rights should be acquired and maintained for a minimum distance along the crossroad, upstream and downstream from the interchange. This "separation distance" is shown in Figure 3-10.

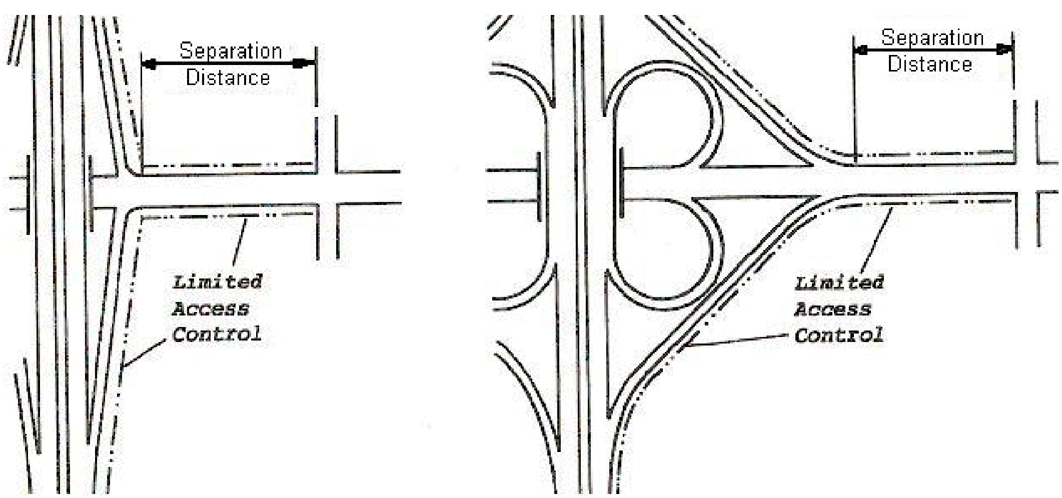


Figure 3-10. Separation Distance for Access Control.

The amount of separation distance needed is dependent on whether the first downstream intersection is signalized or unsignalized. For a signalized intersection, the separation distance should be sufficient for an exit-ramp right-turn vehicle to weave across the crossroad and decelerate into the left-turn bay at the first downstream intersection (or for a vehicle at the intersection to weave across the crossroad and decelerate into the left-turn bay at the interchange ramp terminal). For an unsignalized intersection (or driveway), the separation distance should be sufficient to allow right-turning vehicles to access the adjacent property without disrupting traffic flow through the ramp terminal. The design of the unsignalized access should include channelization to discourage all left-turn movements.

Separation distances for a range of posted speed limits on the crossroad are listed in Table 3-20. The distances shown for unsignalized intersection access are based on the minimum connection spacings listed in Table 2-2 of the *Access Management Manual* (*22*).

Table 3-20. Separation Distance Based on Deceleration and Weaving.

Posted Crossroad Speed, mph	Minimum Separation Distance [1], ft	
	First Signalized Intersection	First Unsignalized Intersection [2]
30	450	200
35	500	250
40	550	305
45	600	360
50	650	425
55	700	425

Notes:
1 - Separation distances are measured along the crossroad from the end of the nearside curb return (or taper) on the ramp to the nearside curb line of the adjacent driveway or intersection.
2 - Raised curb median or island channelization should be used to discourage left-turn maneuvers.

The distances listed for signalized intersection access are based on a synthesis of guidance provided in several references (*10*, *13*, *23*, *24*). Of particular note is the guidance provided on pages 753 and 797 of the *Green Book* (*10*). This guidance indicates that the separation distance should include sufficient length for an exit-ramp vehicle to weave across the crossroad and decelerate into the left-turn bay at the first downstream intersection. It suggests that this distance should be at least 500 ft. Copas and Pennock (*13*) report that the Illinois DOT recommends a minimum separation distance of 500 ft in urban areas and 700 ft in rural areas. The Oregon Department of Transportation recommends a minimum separation distance of 750 ft in built-up urban areas and 1320 ft otherwise (*19*, Chapter 9). The California Department of Transportation recommends a minimum distance of 410 ft for new construction and major reconstruction (*24*, Chapter 500).

CHAPTER 4. CONCLUSIONS

OVERVIEW

Based on a recent change in TxDOT policy, frontage roads are not to be included along controlled-access highways unless a study indicates that the frontage road improves safety, improves operations, lowers overall facility costs, or provides essential access. Interchange ramps in non-frontage-road settings can be more challenging to design than those in frontage-road settings for two reasons. First, they must provide drivers a safe transition between the high-speed freeway and the stop condition at the crossroad intersection. Second, the "effective" ramp length (i.e., that portion of the ramp measured from the gore area to the back of queue) can vary based on traffic demands. Thus, during peak demand hours, the speed change may need to occur over a relatively short length of ramp.

In summary, adequate ramp length, appropriate horizontal and vertical curvature, and flaring to increase storage area at the crossroad intersection might be used to design operationally superior ramps for non-frontage-road settings. However, guidelines for designing ramps in non-frontage-road settings do not presently exist in the *Roadway Design Manual* (*9*). Therefore, a need exists to develop ramp design procedures that address the aforementioned concerns and that can be used to construct safe and efficient interchange ramps for facilities without frontage roads.

The objective of this research project was to develop recommended design procedures for interchange ramps on facilities without frontage roads. The procedures developed describe the operational and safety benefits of alternative ramp configurations. They also offer guidelines that help engineers select the most appropriate configuration for design-year traffic. This chapter documents the findings and conclusions from the research conducted for this project.

SUMMARY OF FINDINGS

Ramp Safety Analysis

In general, safety prediction models relate crash frequency at a facility to its traffic flow, traffic control, and design-related characteristics. They have several uses in safety analysis, including: (1) identifying facilities that may benefit from one or more safety improvements, and (2) providing numerical tools for evaluating alternative design configurations. These models are envisioned to be particularly useful during the concept planning and preliminary design stages of the design process, where they can be used to identify cost-effective interchange ramp configurations.

Interchange safety has been the focus of several previous research projects (*3, 7*). These projects all have in common four ramp configurations that are often used at interchanges without frontage roads; they are:

- diagonal,
- non-free-flow loop,
- free-flow loop, and
- outer connection.

The only safety prediction models specific to ramp type and configuration found in the literature are those developed by Bauer and Harwood (*3*). Their models were developed with data obtained from the State of Washington. Similar models were proposed for development in this research. However, the lack of ramp AADT and the lack of precision in the DPS database in terms of locating ramp-related crashes discouraged this effort. Instead, a more modest database was assembled for the purpose of calibrating the Bauer-Harwood models for use in Texas.

Ramp Crash Characteristics

A database was assembled for the purpose of calibrating the Bauer-Harwood models. It included crash data for 44 ramps at 10 interchanges in Texas. The data were summarized using a variety of statistics for several different categories (e.g., ramp configuration, area type, entrance/exit, etc.). The following observations were made after a review of these statistics:

1. An average of 0.29 crashes per ramp occur each year.
2. Exit ramps have about 62 percent more crashes than entrance ramps, given the same ramp traffic volume.
3. Non-free-flow loop ramps are associated with about twice as many severe crashes as other ramps.
4. The calibrated models indicate the following order of ramp configuration, in terms of decreasing crash rate: non-free-flow loop, outer connection, diagonal, and free-flow loop.
5. Rural ramps have 46 percent more crashes than urban ramps, given the same ramp traffic volume.

With two exceptions, these observations are consistent with those reported in the literature. Observation 3 is contrary to that found in the literature. Other researchers have found that free-flow loop ramps are associated with the *fewest* crashes. It is possible that the high crash rate for free-flow loops in Texas is related to driver familiarity. These loops are not currently used as often in Texas as in other states.

For Observation 5, the trend for rural ramps to have more crashes than urban ramps is logical given the higher speeds found at rural interchanges and their general lack of safety lighting. However, it is contrary to that reported by Bauer and Harwood (*3*). Closer examination of the data indicated that a majority of the PDO crashes occurring on the urban ramps are not being reported. After accounting for unreported crashes, rural ramps in Texas were found to have only about 45 percent more crashes than urban ramps.

Procedure for Comparing Alternative Interchange Types

A four-step procedure was developed for comparing alternative interchange types and ramp configurations in terms of their expected crash frequency. The procedure is based on a series of safety prediction models calibrated to Texas conditions. The procedure can be used to predict the expected annual crash frequency for a specified ramp configuration. This estimate can then be aggregated to obtain an estimate for the entire interchange. The analysis steps include:

- Identify area type and movement AADTs.
- Identify candidate ramp configurations for each interchange quadrant.
- Estimate annual crash frequency for each ramp alternative.
- Estimate interchange crash frequency.

Techniques are provided for: (1) estimating ramp turn movement AADTs based on the major-road AADT, and (2) interpreting the crash statistics obtained from the evaluation.

Ramp Selection Guidelines

Ramp design guidelines were developed for freeway facilities without frontage roads. Design controls and elements routinely considered during the ramp design process are identified. This guidance is presented in a more concise manner in Report 0-4538-3, *Recommended Ramp Design Procedures for Facilities without Frontage Roads*.

The ramp design guidelines were developed from a variety of sources. These sources include: design guidelines used by several state DOTs, interviews with TxDOT engineers, analysis of data obtained from field and simulation studies, analysis of crash data for interchanges in Texas, and the guidance in the *Roadway Design Manual* (*9*) and in *A Policy on Geometric Design of Highways and Streets* (*Green Book*) (*10*).

Alternative Interchange Types

The two most common types of service interchange for non-frontage-road settings are the diamond and parclo. Typical variations of both types are shown in Figure 1-1. The absence of frontage roads allows for the consideration of a wide range of interchange types. This range allows for a more cost-effective ramp configuration to be selected in terms of its accommodation of site-specific traffic demands, topographic features, and right-of-way constraints.

One advantage of the diamond interchange is that the turn movements from the major road and from the crossroad are "true" to the intended change in direction of travel. In other words, a driver makes a left turn at the interchange when desiring to make a left turn in travel direction. This characteristic is desirable because it is consistent with driver expectancy.

In urbanized areas, the tight urban diamond interchange and single-point urban interchange can provide efficient traffic operation along the crossroad. The tight urban diamond interchange's ramp terminals are easily coordinated using a single signal controller. The efficiency of the single-point urban interchange stems from its use of a single signalized junction and non-overlapping left-turn paths. In contrast, the compressed diamond is not as operationally efficient as the tight urban diamond interchange or single-point urban interchange. This characteristic is due to the compressed diamond's wider ramp separation, which is not as conducive to crossroad signal coordination.

The parclos shown in Figure 1-1 are most applicable to situations where a specific left-turn movement pair has sufficiently high volume to have a significant negative impact on ramp terminal operation. Both variations of the parclo A provide uncontrolled service for crossroad drivers intending to turn left (in travel direction) at the interchange. Both variations of the parclo B typically provide uncontrolled service for major-road drivers intending to turn left at the interchange.

The parclo A and parclo B are more efficient than their "2-quad" counterparts. This feature stems from their elimination of one left-turn movement from the ramp terminal signalization. One advantage of the parclo A is that it satisfies the expectancy of major-road drivers by providing turn movements that are "true" to the driver's intended direction of travel. A second advantage is that it does not require left-turn bays on the crossroad. This advantage can result in a narrower bridge. A third advantage is that its terminal design is not conducive to wrong-way movements. Finally, the speed change from the crossroad to the loop ramp is likely to be relatively small and easy to accommodate with horizontal curves of minimum radius.

The parclo B also has several advantages. One advantage is that its signalized ramp terminals do not require coordination because signal timing for the outbound travel direction can provide a continuous green indication. A second advantage is that it does not require queues to form on the exit ramp because the left-turn and right-turn movements are unsignalized at their intersection with the crossroad. Finally, its ramp terminal design is not conducive to wrong-way movement.

Overpass vs. Underpass

A fundamental consideration in interchange design is whether the major road should be carried over (i.e., an overpass design) or under the crossroad (i.e., an underpass design). When topography does not govern, it appears that the underpass design offers greater benefit when ramp safety and operations are key considerations. The underpass design with the crossroad elevated "above" ground level is often the most advantageous because it provides major-road drivers with: (1) ample preview distance as they approach the interchange, and (2) ramp grades that are helpful in slowing exit-ramp drivers and accelerating entrance-ramp drivers.

Selection Considerations

Interchange selection should reflect consideration of safety, operation, uniformity of exit patterns (relative to adjacent interchanges), cost, availability of right-of-way, potential for stage construction, and compatibility with the environment (*10*). The selection of interchange type for rural areas is based primarily on traffic demand, especially turn movement demands. In urban areas, selection is based on traffic demands, interchange spacing, and right-of-way impacts. Interchanges with loop ramps can be very efficient at locations with heavy left-turn volumes; however, their right-of-way requirements can preclude them from consideration in built-up urban environments.

Procedures for evaluating the safety and operation of interchange types have been developed for this research. The procedures are sufficiently general that they can be used for the selection of ramp configuration and interchange type for the concept planning and preliminary design stages of the design process. The safety evaluation procedure is described in Chapter 2. The operations evaluation procedure is described by Bonneson et al. (*4*).

Ramp Design Guidelines

Ramp Segment Design Speed

A procedure is developed for the effective use of design speed as a control in ramp design. The approach is tailored to the ramp's configuration such that design speed changes are gradual and consistent with driver expectancy and operational capabilities. It is based on the specification of reasonable speed changes along the various tangents and curves that compose the ramp's horizontal alignment. The total amount of speed change needed is dictated by the design speed of the major road and that of the intersecting crossroad or ramp terminal. This approach is consistent with the ramp design speed guidance provided on page 829 of the *Green Book* (*10*)

The procedure is based on the separate consideration of the tangent and curve segments that compose the interchange ramp alignment. Rules are cited for defining a reasonable design speed for each segment. These rules were used to develop the ramp segment design speeds in Table 3-6. The analyst is encouraged to use the cited rules to identify segment design speeds for situations not addressed by this table.

Horizontal Geometrics

Design controls are described for determining the minimum length of the various ramp segments as well as the overall length of the ramp. These controls are listed in Table 4-1. The guidelines provided describe how the various basic control values should be selected and used to compute or define the relevant design element controls.

Cross Section

Guidance on topics related to ramp cross section design were synthesized from the literature. This guidance addresses the number of lanes on the ramp and the width of the ramp traveled-way. A single-lane ramp cross section should be adequate for most service interchanges. However, five conditions are cited that, if satisfied, may indicate the need for a dual-lane ramp.

Table 4-1. Design Controls Considered in Ramp Design.

Ramp Length Consideration	Basic Controls	Design Element Controls
Vertical Alignment	• Controlling curve design speed • Major-road design speed	• Maximum grade
Speed Change and Storage	• Design volume • Major-road design speed • Crossroad ramp terminal design speed	• Minimum deceleration length • Minimum storage length • Minimum acceleration length
Segment Design Speed	• Major-road design speed • Crossroad design speed • Segment design speed	• Minimum travel time • Minimum super. transition length • Minimum deceleration length • Minimum acceleration length • Maximum superelevation rate • Minimum radius

Ramp traveled-way widths should be based on ramp curvature, number of lanes provided, and the portion of trucks in the traffic stream to accommodate truck off-tracking. Desirable traveled-way widths for ramps with curbs or shoulders are provided in Table 3-17. The values in this table indicate that ramp widths in excess of 14 ft may be needed if the percentage of heavy vehicles exceeds 10 percent, a curb is used on one or both sides of the ramp, or if the curve radius is less than 500 ft.

Ramp Terminal Design

Guidance on topics related to ramp terminal design have also been synthesized from the literature. These topics include: ramp terminal traffic control, intersection skew angle, departure leg design, approach leg design, design to discourage wrong-way maneuvers, and access control on the crossroad. Ramp terminal design for non-frontage-road settings can include a discontinuous alignment through the ramp/crossroad junction. This discontinuity allows each ramp junction leg to be skewed in a direction toward the major road, thereby minimizing the curvature on the ramp proper. However, this feature is not seen to offset the disadvantages of a skewed intersection alignment. Desirably, the alignment of the ramp would be designed such that skew is avoided at the ramp terminal.

The departure leg of the entrance ramp should accommodate the swept path of the design vehicle as it negotiates a left turn or a right turn from the crossroad. Specifically, the traveled-way within the throat of the departure leg should be sufficiently wide as to allow the design vehicle to turn from the crossroad and enter the ramp without, at any point, encroaching on an adjacent lane, shoulder, or curb.

The number of lanes provided on an exit-ramp terminal approach should reflect consideration of the traffic control mode, turn volumes, and crossroad through volume. The guidance provided in the discussion associated with Table 3-10 can be used to determine the minimum number of lanes needed on the ramp terminal approach. As a minimum, it is preferable to provide at least one approach lane for each traffic movement served at the ramp terminal. This treatment permits the use of separate traffic control modes to regulate each movement and should minimize delays and queues to ramp traffic.

The control of access along the crossroad is essential to the safe and efficient operation of the interchange. The importance of access control is heightened when frontage roads are not provided because of the inherent increase in turning traffic and the focus on development of properties adjacent to the crossroad. Inadequate access control in the vicinity of the interchange can create operational problems on the crossroad that may propagate to the ramps causing spillback onto the major road. To ensure efficient interchange operation, access rights should be acquired and maintained for a minimum "separation distance" along the crossroad, upstream and downstream from the interchange.

The amount of separation distance needed is dependent on whether the first downstream intersection is signalized or unsignalized. For a signalized intersection, the separation distance ranges from 450 to 700 ft. The lower value in this range is associated with a posted crossroad speed of 30 mph; the upper value is associated with a speed of 55 mph. For an unsignalized intersection (or driveway), the separation distances range from 200 to 425 ft for speeds of 30 to 55 mph.

CONCLUSIONS

The objective of this research project was to develop recommended design procedures for interchange ramps on facilities without frontage roads. This objective was achieved in part by developing procedures for evaluating the operation and safety of alternative ramp configurations. The remaining part of the objective was achieved by the development of ramp design guidelines.

A wide variety of interchange types and associated ramp configurations can be used on freeways without frontage roads. This variety provides additional flexibility to the designer by allowing him or her to adapt the ramp configuration to the existing topography and right-of-way without any compromise in safety and efficiency. The parclo family of interchanges as well as the single-point urban interchange can provide cost-effective solutions in specific situations. The evaluation procedures developed in this research can be used to identify the settings where these or other interchanges are viable from a safety and operations standpoint.

There is considerable guideline information available nationally that describes controls and considerations for interchange ramp design for non-frontage-road settings. This information has been synthesized and modified in this research and recast as guidelines applicable to Texas design practice. These guidelines have been extended in this research by the development of a procedure for specifying a design speed for each ramp segment. This segment-based approach to ramp design ensures a uniform and consistent sizing of ramp geometry such that it matches the expectation of the ramp driver.

CHAPTER 5. REFERENCES

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APPENDIX

SAFETY DATA COLLECTION SITES

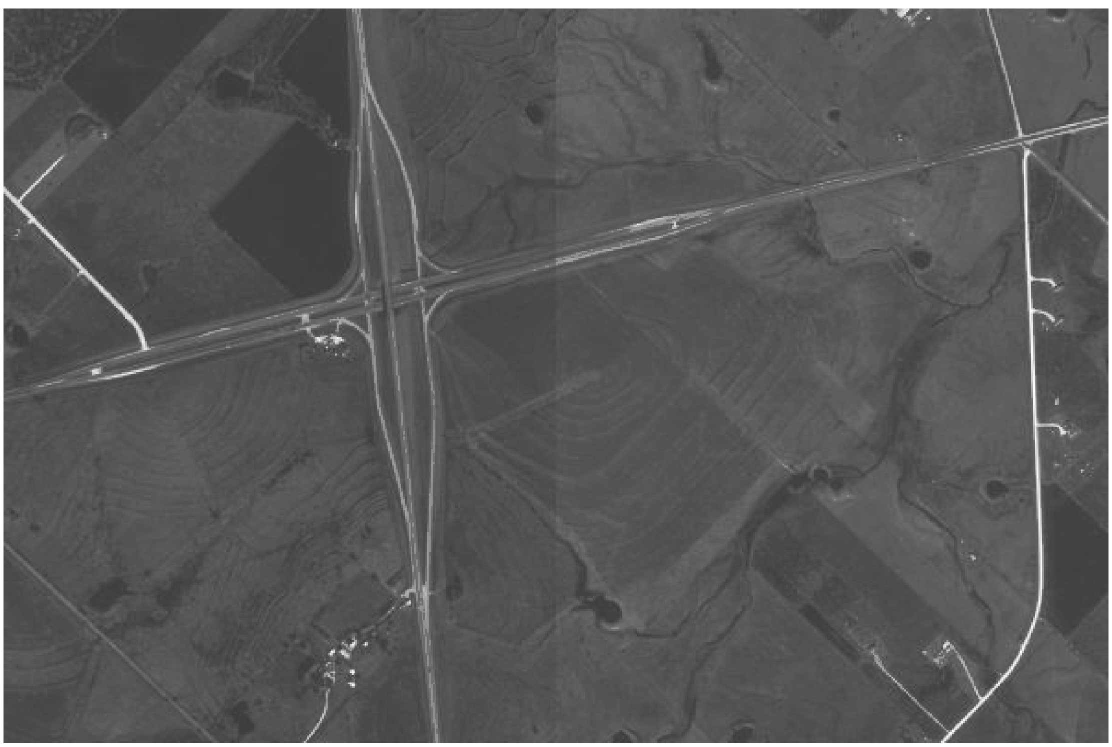
Table A-1. Data Collection Sites.

County	Address	Terminal Control [1]	Area Type	No. of Sites for Each Ramp Configuration [2, 3]			
				Diagonal	NFF Loop	FF Loop	Outer
Bastrop	US 290 & SH 21	TWSC	Rural	4	0	0	0
Caldwell	US 183 & SH 21	TWSC	Rural	4	0	0	0
Lee	SH 21 & US 77	TWSC	Rural	4	0	0	0
Travis	Loop 360 & RM 2222	Signal	Urban	4	0	0	0
	US 183 & FM 969	Signal	Urban	4	0	0	0
	US 183 & SH 71	Free	Rural	0	0	3	4
	SH 1 & Windsor	Signal	Urban	1	2	0	0
	SH 1 & 35th Street	Sig./TWSC	Urban	1	1	2	2
	Loop 360 & RM 2244	Signal	Urban	4	0	0	0
Williamson	US 79 & Business 79	TWSC	Rural	0	0	2	2
Total Candidate Sites:				26	3	7	8

Notes:
1 - TWSC: two-way stop control; Signal: traffic control signal; Free: merge from a parallel or tapered ramp terminal.
2 - NFF Loop: non-free-flow loop; FF Loop: free-flow loop.
3 - A "site" is one interchange ramp.

1. US 290 & SH 21

2. US 183 & SH 21



3. SH 21 & US 77

4. Loop 360 & RM 2222

5. US 183 & FM 969

6. US 183 & SH 71

7. SH 1 & Windsor

8. SH 1 & 35th Street

9. Loop 360 & RM 2244

10. US 79 & Business 79